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Volume I
Analytical Studies

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EPA-600/2-76-217a
September 1976

URBAN RUNOFF CHARACTERISTICS

Volume I - Analytical Studies

by

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U.S. ENVIRONMENTAL PROTECTION AGENCY
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TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/2-76-217a	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE URBAN RUNOFF CHARACTERISTICS - Volume I - Analytical Studies		5. REPORT DATE September 1976, Issuing Date 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Herbert C. Preul and C. N. Papadakis		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Division of Water Resources Department of Civil & Environmental Engineering University of Cincinnati Cincinnati, Ohio 45220		10. PROGRAM ELEMENT NO. 1BC611 11. CONTRACT/GRANT NO. 11024-DQU
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED Final 14. SPONSORING AGENCY CODE EPA-ORD

15. SUPPLEMENTARY NOTES

P. O. - Anthony N. Tafuri, 201/548-3347 Ext. 512
See Volume II, EPA-600/2-76-217b

16. ABSTRACT

This is a report on analytical studies and field investigations for the characterization of urban runoff and combined sewer overflows. The report is divided into two volumes as follows:

Volume I - Analytical Studies
Volume II - Field Investigations

Volume I deals mainly with the analysis and development of stormwater management models. Initially, the results of studies are given on the development of storm hyetographs and the determination of infiltration capacity curves for use in hydrologic runoff models. In the next sections, an hydrologic model known as "the University of Cincinnati Urban Runoff Model" is developed and a general description is given of the EPA Storm Water Management Model. Results are then provided on the testing of these computer models using field data included in Volume II.

Volume II is a compilation of hydrologic and water quality data collected over a three year period from a typical combined sewer watershed of 2380 acres area in Cincinnati, Ohio.

17. KEY WORDS AND DOCUMENT ANALYSIS

PRICES SUBJECT TO CHANGE

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
*Runoff, *Overflows, Model Tests, *Computer Programs, *Field Investigations, Sampling	Storm Water, Storm Water Management Models, Computer Model Testing, Field Investigations, Urban Watersheds, Hydrographs, Pollutograph, Management Models, Runoff Characteristics	08H 09B 13B 14B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 353
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE $10-50-3.00

EPA Form 2220-1 (9-73)

DISCLAIMER

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report focuses on the characterization of liquid contaminants which overflow from combined sewer systems. It involves a typical combined sewer watershed in Cincinnati, Ohio.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

This is a report on analytical studies and field investigations for the characterization of urban runoff and combined sewer overflows. The report is divided into two volumes:

Volume I - Analytical Studies

Volume II - Field Investigations

Volume I deals mainly with the analysis and development of stormwater management models. Initially, the results of studies are given on the development of storm hyetographs and the determination of infiltration capacity curves for use in hydrologic runoff models. In the next sections, a hydrologic model known as "the University of Cincinnati Urban Runoff Model" is developed and a general description is given of the EPA Storm Water Management Model. Results are then provided on the testing and verification of these computer models using field data included in Volume II.

In the final two sections of Volume I, a general analysis is made of the field data on sewer overflows as given in Volume II and these data have then been further utilized in determining the required retarding basin capacity for a combined sewer watershed.

Volume II is a compilation of hydrologic and water quality data collected over a three year period from a typical combined sewer watershed of 2380 acres area in Cincinnati, Ohio.

This report was submitted in fulfillment of Project No. 11024 DQU by the Division of Water Resources, Department of Civil and Environmental Engineering of the University of Cincinnati, under the sponsorship of the U.S. Environmental Protection Agency.

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ACKNOWLEDGMENTS

A list is given below of the various people in the Division of Water Resources at the University of Cincinnati who have contributed significantly to this research project and the production of this report.

Principal Investigator & Project Director	Dr. Herbert C. Preul Associate Professor of Civil & Environmental Engineering
Acting Project Director, Summer, 1970	Dr. L. M. Laushey, Professor & Head, Dept. of Civil & Environmental Engineering
Project Engineer & Coordinator	Constantine Papadakis & Jeffrey D. Sharon
Laboratory Supervisor	D. D. Dasnurkar
Field & Office Engineers	Paul F. Carlson David L. Maase R. T. Kuo Lester M. A. Gulitz Keith Malcolm James Gutzwiller T. S. Chen C. S. Lu J. H. Tay A. S. Rashidi T. C. Wu J. S. Chien J. B. Malouf

Appreciation is extended to the following officials of the United States Environmental Protection Agency for their assistance and cooperation during the course of the project.

Mr. Robert L. Feder	Project Officer, 1st Year, EPA
Mr. Anthony N. Tafuri	Project Officer, 2nd & 3rd Years, Storm & Combined Sewer Section, EPA
Mr. Richard Field	Chief, Storm & Combined Sewer Section, EPA
Mr. Darwin R. Wright	Acting Director, Agriculture & Non-Point Sources Management Division, EPA
Mr. William A. Rosenkranz	Director, Waste Management Division, EPA

SECTION 1

CONCLUSIONS AND RECOMMENDATIONS

Two significant efforts have been carried out in connection with this project. One effort concentrated on analytical studies for the development of stormwater management models and another effort concentrated on the collection of field data for the characterization of urban runoff and storm sewer overflows. The conclusions and recommendations resulting from the project are divided accordingly with headings "Analytical Studies" and "Field Investigations".

ANALYTICAL STUDIES - CONCLUSIONS AND RECOMMENDATIONS

A number of mathematical models for high speed computation of urban runoff quantity and quality have been developed. Perhaps the most comprehensive of these models is the U. S. Environmental Protection Agency's Storm Water Management Model (SWMM). Other models are available which provide for only the calculation of hydrologic discharges. A deterministic model of this type is the University of Cincinnati Urban Runoff (UCUR) Model. The following conclusions and recommendations were derived from working with these models and their related input parameters.

CONCLUSIONS (Analytical Studies)

1. Both the SWMM and UCUR models were found to give reasonably acceptable accuracy for the calculation of runoff hydrographs when compared with actual field data. However, the input parameters must properly characterize the sub-areas within the watershed. Two of the most important inputs are rainfall data and surface infiltration. Variations in these parameters will substantially affect the hydrograph results for both of these models and also the pollutographs for the SWMM calculation of water quality.

2. In using the models on larger watersheds, there is less chance that a particular storm will cover the area uniformly, therefore in comparing the computed output with actual field observations, the actual coverage and movement of the storm may be a source of inaccuracy in developing the hyetograph from rainfall observations. These factors appeared to be significant possible sources of inaccuracy in the results.

3. In the operation of urban runoff models for either existing or design purposes, it is necessary to select a rainfall hyetograph which is representative of the conditions of concern. This may be an observed storm or it may be a selected storm of a desired frequency. Since usually only an experimental watershed would have automatic rainfall gaging, as in the case of this project, it becomes necessary to utilize rainfall intensity data from a central weather observation station which may be generally representative of local conditions. Therefore, an approach has been given in this report which may be useful in the development of design storm hyetographs of selected frequencies for use in mathematical models.

4. Surface infiltration was found to be a model input parameter of vital importance in resulting accuracy and it is of increased importance for watershed areas having large pervious portions.

5. Since the models rely on the infiltration capacity curves to reflect the watershed moisture conditions, it is important that this input be representative. For this reason, preliminary work was carried out under this project on the determination of infiltration capacity curves. The purpose of the investigations was to relate Horton's initial rate of infiltration and decay rate of infiltration with the antecedent precipitation. A method has been proposed which allows the use of Horton's infiltration equation for a certain soil type and for specified antecedent conditions.

6. From an overall comparison of working with the models, a greater degree of confidence in results can be expected for smaller areas. For larger watershed areas, larger departures in accuracy from the actual, seemed to result for both the computed hydrographs and the computed pollutographs. This indicates that the accuracy of subdivision of the total area into sub-catchment areas is important and that accuracy should improve with more extensive subdivision.

7. The UCUR model was found useful for calculating urban runoff hydrographs with an acceptable degree of accuracy. Testing results have shown that it compares favorably with that of more sophisticated models which require more detailed data and computer time. However, in using the model, its limitations should be kept in mind

regarding flow routing and storage within the system or in the form of depression storage on the watershed.

8. Since the retardation of urban runoff is one approach for controlling urban runoff and combined sewer overflows, a method has been developed for the stochastic design of retarding basins which may be used in the storage of combined sewer overflows. Field data from a typical watershed have been used to illustrate the method.

RECOMMENDATIONS (Analytical Studies)

1. Although the models worked with in this project offer a comprehensive degree of simulation of the stormwater runoff process, particularly the EPA SWMM, it is apparent that rather extensive engineering specialization is necessary in order to operate the models and rather detailed field data are required as input, therefore it is recommended that study be given to appropriate simplifications which will make the models more widely acceptable in practice.

2. On the basis of these studies, it is recommended that further research be carried out to better characterize the more significant input parameters such as the rainfall hyetograph and surface infiltration.

3. Research is recommended on aspects of watershed surface ponding and flooding which need to be better incorporated into the models to improve their accuracy. Such information also is of importance for flood control purposes in urban areas where many floods occur due to inadequate drainage systems.

4. It is recommended that research be supported for further study of the selection of hyetographs for runoff models, in particular with regard to frequency relationships.

5. In the future researching of the rainfall intensity input for the models, it is recommended that studies be made of the effects of storm patterns and movement across a watershed, particularly for the larger watersheds.

6. In the future researching of the infiltration input for the models, it is recommended that further attention be given to the evaluation of antecedent moisture conditions as related to infiltration.

FIELD INVESTIGATIONS-CONCLUSIONS AND RECOMMENDATIONS

Discharge measurements and sequential sampling of water quality have been carried out at five sewer locations, including the outlet, in the combined sewer system of the Bloody Run Sewer Watershed in Cincinnati, Ohio, over a three-year period from 1970 to 1972. Runoff quantity and quality data, along with detailed physical information on the sewer system and watershed, for the first-year operations up to October, 1970, have been published in the first-year interim report entitled "Urban Runoff Characteristics", Water Pollution Control Series 11024 DQU 10/70, by the Water Quality Office of the Environmental Protection Agency.

During the three years of the project, detailed hydrologic and water quality observations were made at five locations in the watershed for 140 rainstorms. The data collected included:

- precipitation hydrograph measurements for each storm at automatic stations set up for three locations within the watershed;
- discharge hydrograph measurements at five sewer locations within the watershed;
- runoff water quality samples collected on a sequential basis and analyzed for BOD, COD, suspended solids (total and volatile), total coliform, chloride, and pH at five sewer locations within the watershed; and
- detailed measurements and analyses of the quantity and quality of dry weather flow.

As a result of this work and experience, the following conclusions and recommendations were derived.

CONCLUSIONS (Field Investigations)

1. The collected data in Volume II have value in that they provide results over a three-year period on the monitoring of combined sewer flows at points within a system as well as at the outlet. The data have been tabulated on a detailed time basis and have also been developed into the form of plots of hyetographs, hydrographs and pollutographs so that comparisons can be made for various purposes. For example, the data may be used in making general comparisons with other watersheds for certain predictions or evaluations. Use may also be made in testing the validity of storm-

water management models, as in this project.

2. Because the intensity of certain storms may vary over the area of a watershed, particularly a large one, it is desirable to have an experimental watershed amply covered with automatic rainfall gages. Although three antomatic gages were used to triangulate the 2380-acre watershed in this project, and were indicated to be adequate for storms of a general type, this observation coverage appeared to be inadequate for certain storms which varied significantly in their pattern and intensity in moving across the watershed. In the model testing and comparisons of computed results with actual for certain storms, this was indicated to be a possible source of deviation in the results.

3. Hydrologic events are of a random nature and rainfall often occurs over an area with limited advance indications, therefore for the experimental purposes of this project, it was necessary to employ automatic equipment for gaging and sampling at points within the sewer system as well as at the outlet. The methods used are indicated to have given reasonably accurate results for purposes of characterizing the flows from a typical watershed. However, the sampling equipment in some cases was a limitation in obtaining complete data for a storm because it allowed sampling only up to a two-hour period and in some cases malfunctions of the equipment occurred, therefore, some improvements in the sampling methods on future such projects would be desirable.

RECOMMENDATIONS (Field Investigations)

1. It is recommended that experimental urban watersheds, of a typical type, be set up for continuous flow and water quality monitoring in representative regions of the country so that a background of data over extended periods of time can be developed both for research purposes and for use in practice. Only very limited data, such as given in this report, are currently available. In order to study hydrologic events and effects for urban watersheds, more extensive data are seriously needed.

2. Although the aspects of water quality for an urban watershed were emphasized under this project, the flood control aspects are also of considerable importance and it is recommended that future field investigations in urban watersheds also emphasize this concern. Urban flooding is an increasing problem in urban areas because the basic drainage systems in many cases were constructed many years ago for a lower intensity of development and for lower frequency

storms than may now be appropriate.

3. For a thorough observation of rainfall intensity during storms over an extensive urban watershed area, it is recommended that sufficient gages be provided so as to represent subdivisional areas not larger than several hundred acres. This will allow a close monitoring of the rainfall intensity with respect to subdivisional areas. It also will allow an analysis of the pattern and movement of storms which are not general in nature.

4. In future field investigations of urban watersheds for experimental purposes, it is recommended that water quality monitoring and recording equipment be used which will provide continuous sensing of the quality of stormwater flows rather than to use sequential sampling as in this project. Although, the initial cost may be higher for the installation of equipment, this approach would allow more extensive data and would relieve the burdensome task of analyzing large numbers of samples. It is also recommended that such a provision would include the central readout of data at a convenient remote location for continuous monitoring and observations.

5. Current methods of flow measurement, within a combined sewer system and at the outlet, limit the accuracy of flow measurements under certain conditions and it is therefore recommended that research be directed to improve this aspect of monitoring.

SECTION 2

INTRODUCTION

This is a report on urban runoff investigations carried out over a three-year period from 1970 to 1972 under Research Grant 11024 DQU to the University of Cincinnati by the U.S. Environmental Protection Agency, Office of Research and Development.

The general objective of the project was to gather data for estimating the magnitude, frequency, and quality of combined sewer overflows from urban watersheds.

The main effort during the first year was to establish a monitoring system and to gather as much field data as possible on a typical urban watershed. The monitoring system which was established differs from various systems that have been set up for past studies in that it provides for observations of the rates of runoff (quantity and quality) at a number of locations within the sewer system as well as at the outlet of the drainage area. Most of the past studies have included observations only at the outlets. In addition, considerable effort was given to the development of mathematical models which would simulate and control a storm-water overflow system.

The second and third-year programs concentrated on the gathering of field data to a maximum extent on the existing combined sewer watershed.

Testing and verification of the mathematical models have been carried out during the third year with available data. These models include the Storm Water Management Model furnished by EPA (developed by Metcalf and Eddy, Inc. and others under their contracts 14-12-501, 502, 503) and also a model developed by the University of Cincinnati.

SECTION 3

DEVELOPMENT OF DESIGN STORM HYETOGRAPHS FOR USE IN MATHEMATICAL MODELS

SYNOPSIS

In order to use deterministic mathematical models, such as the EPA Storm Water Management Model and the University of Cincinnati Urban Runoff Model, for the simulation of urban runoff, it is necessary to input appropriate information on the characteristics of the watershed and any selected rainfall hyetograph. However, if these models should be used for the design and analysis of sewer systems, the input rainfall should be a design rainfall hyetograph for a desired storm frequency. This section provides information on the development of such a hyetograph.

Synthetic design storm patterns of selected frequencies are presented for a Cincinnati, Ohio watershed. A similar approach may be applied to other locations. In general, the procedure involves establishing rainfall intensity-duration curves for selected design frequencies for a particular location and then deriving corresponding storm patterns using a method first proposed by Keifer and Chu [3] and later implemented by Bandyopadhyay [1].

The location of the peak of the synthetic storm pattern is established by relating to antecedent precipitation. For Cincinnati, the required antecedent precipitation data were obtained from detailed hyetograph records, collected from three automatic raingages located in the vicinity of the Bloody Run Sewer watershed.

INTENSITY-DURATION CURVES FOR CINCINNATI, OHIO, FOR SELECTED DESIGN FREQUENCIES

Rainfall intensity-duration relationships for a given frequency and less than a two-hour duration can be expressed mathematically [5] by an equation of the form:

$$i_{av} = \frac{a}{(t_d+b)^c} \tag{1}$$

where i_{av} = average intensity in inches/hour

t_d = duration of the maximum period of rainfall, in minutes

a,b,c = constants to be computed for each frequency N.

The method for determining the constants a, b and c is presented for the 1-year frequency rain and the values of these constants corresponding to the 2, 5, 10, 25, 50 and 100-year frequencies are listed in Table 3-2.

For each frequency N, the 1-hour duration rainfall in inches, to be expected at Cincinnati once in N years was obtained from the corresponding isopluvial map of the U.S.A. [4] and was tabulated in Table 3-2. As an illustration the 1-year, 1-hour rainfall isopluvial map of the U.S.A. is presented in Figure 3-1, from which the mass of the 1-hour rainfall to be expected at Cincinnati once a year is 1.10 inches. The average intensity of that rainfall which has a duration of one hour (t_d = 60 min.) is i_{av} = 1.10 in/hr. As a check, each frequency and the corresponding 1-hour rainfall (tabulated in Table 3-2) were plotted on semi-logarithmic paper in Figure 3-2. The result was a straight line as expected.

The generalized rainfall intensity-duration curves prepared by the U.S. Army Corps of Engineers [2] are presented in Figure 3-3. Curve numbers correspond to 1-hour values of rainfall in inches. Thus, these curves are readily applicable to most geographic locations in the U.S.A. For example, the No. 1.8 curve is approximately the 10-year frequency curve for Ann Arbor, Michigan. However, the same curve is roughly the 7-year frequency curve for Cincinnati, Ohio. It should be pointed out that conventional intensity-duration-frequency curves correspond to frequencies and are derived for particular locations. If intensity-duration-frequency curves are available for the area under study, their use should substitute the isopluvial maps and the generalized intensity-duration curves.

Making use of the generalized rainfall intensity-duration curves of Figure 3-3, the point with coordinates t_d = 60 min. and i_{av} = 1.1 in./hr. falls on the No. 1.1 curve. The fact that all points on the same curve have the same average frequency of occurrence can be checked by finding the mass of the 30-minute rainfall to be expected at Cincinnati once in one year which is 0.92 inches. The average intensity of that rainfall is $i_{av} = 0.92 \times 60/30 =$ 1.85 in./hr. The point with coordinates t_d = 30 min. and i_{av} = 1.85 in./hr. falls on the No. 1.1 curve of Figure 3-3 as should be expected.

The values of a, b and c for N = 1 year were obtained by fitting equation (1) to the No. 1.1 curve of Figure 3-3.

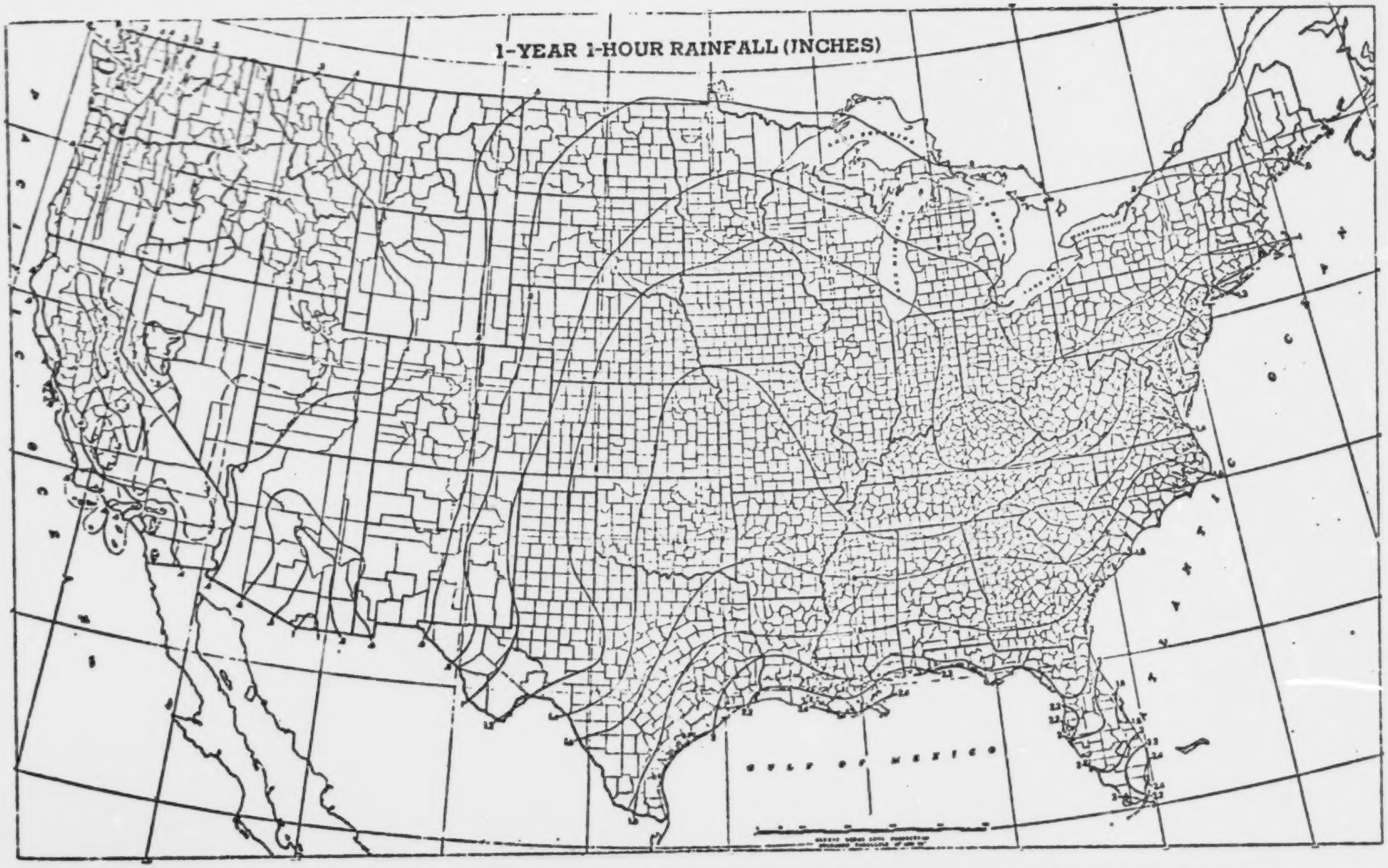
1-YEAR 1-HOUR RAINFALL (INCHES)
PACIFIC OCEAN
ATLANTIC OCEAN
GULF OF MEXICO

Figure 3-1. ISOPLUVIAL MAP, USA, 1-YEAR FREQUENCY,1-HOUR DURATION RAINFALL(INCHES) [3]

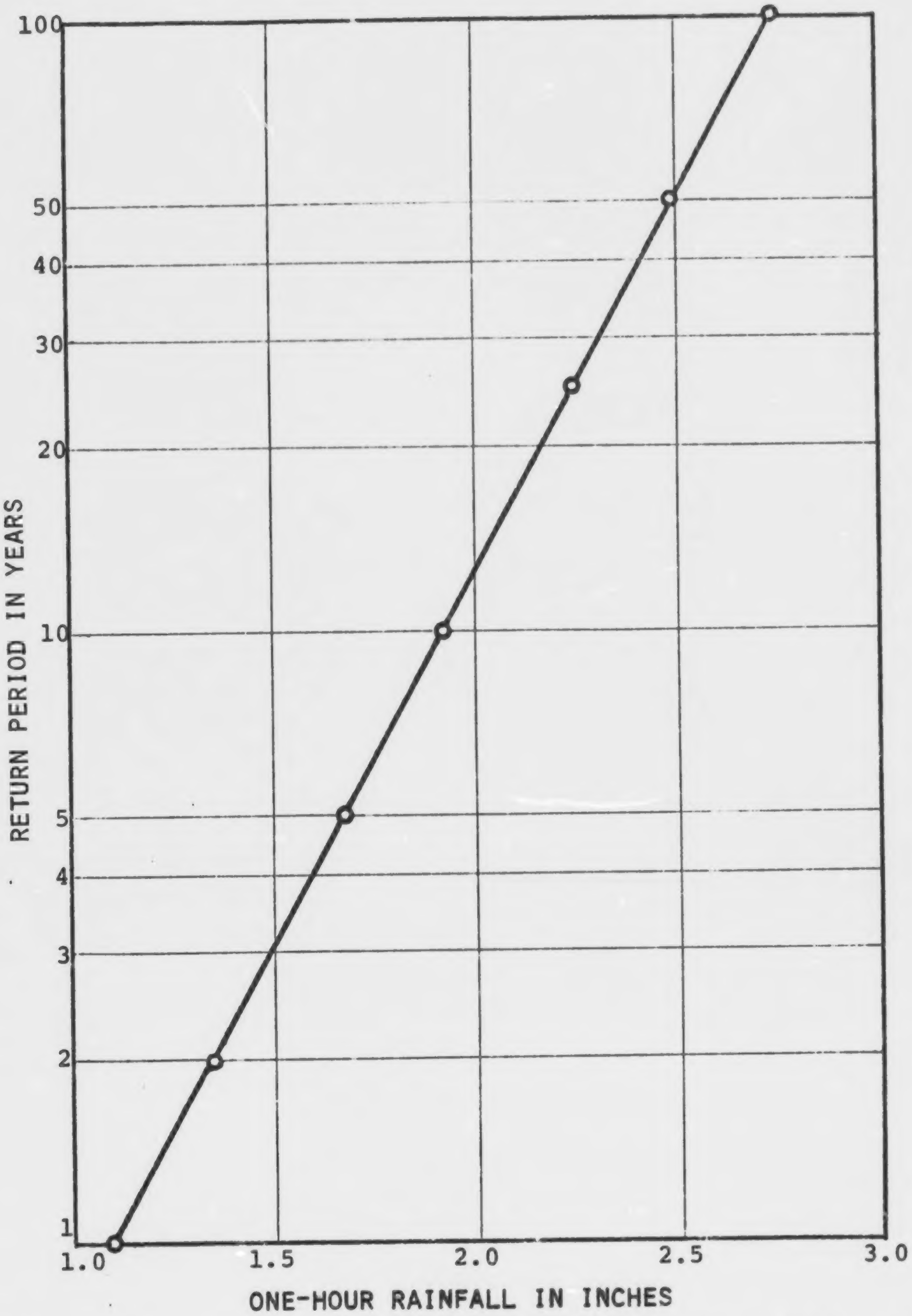

Figure 3-2. FREQUENCY VS 1-HR RAINFALL FOR CINCINNATI, OHIO

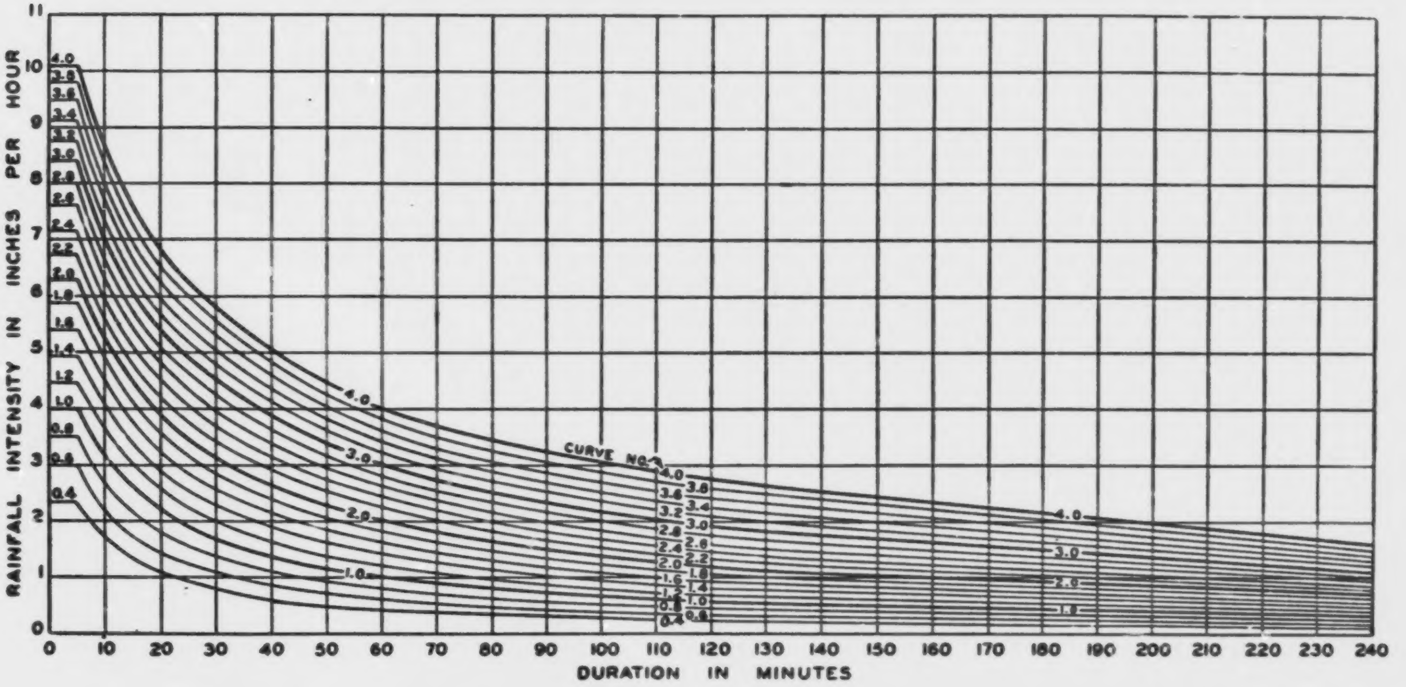

Figure 3-3. GENERALIZED RAINFALL INTENSITY-DURATION CURVES. [2]
(CURVE NUMBERS CORRESPOND TO 1-HR. VALUES OF RAINFALL IN INCHES)

Table 3-1. DETERMINATION OF THE INTENSITY-DURATION CURVE WITH 1-YEAR FREQUENCY FOR CINCINNATI, OHIO.

Duration t_d(min)	Av. rainfall* i_{av}(in/hr)	$t_d + b$		
		b = 5	b = 10	b = 20
5	4.50	10	15	25
10	3.50	15	20	30
20	2.40	25	30	40
30	1.85	35	40	50
40	1.50	45	50	60
60	1.10	65	70	80
80	0.87	85	90	100

*For a 1-year frequency, extrapolated from Curve No. 1.1 of Figure 3-3.

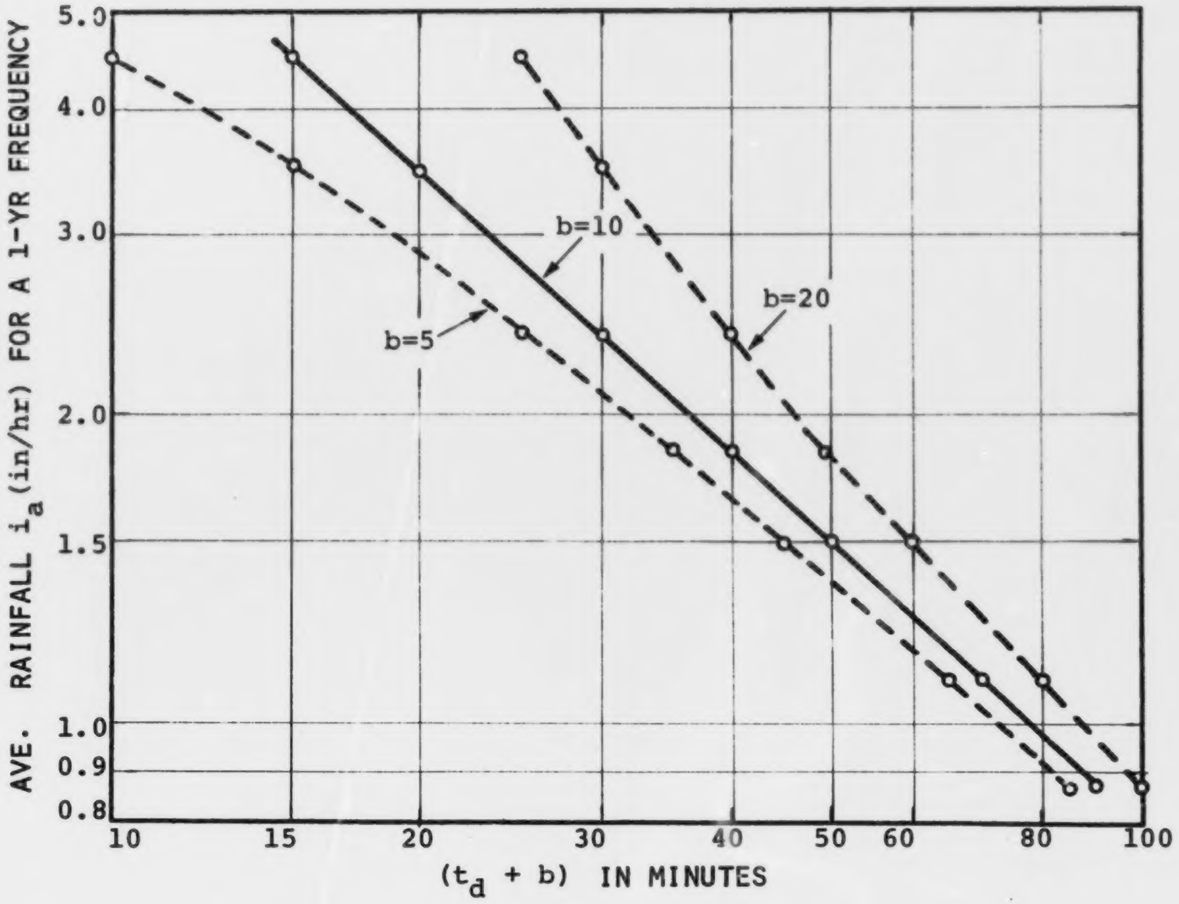

Figure 3-4. DETERMINATION OF THE INTENSITY-DURATION CURVE WITH 1-YEAR FREQUENCY FOR CINCINNATI, OHIO.

Equation (1) can be written in logarithmic form as:

$$\log i_{av} = \log a - c \log (t_d+b)$$

in which, if b is selected properly, the logarithms of i_{av} and (t_d+b) should plot as a straight line having a slope equal to -c. The value of b was determined by plotting points with various assumed values of b until a straight line was established, after which the intersect a and the slope -c were found graphically [5]. This procedure is presented in Table 3-1 and in Figure 3-4 and resulted in a = 53, b = 10 and c = 0.91. Therefore, at Cincinnati the intensity-duration curve for a 1-year design storm is:

$$i_{av} = \frac{53}{(t_d+10)^{0.91}}$$

Table 3-2 presents in tabulated form the constants a, b and c of equation (1) for different frequencies for the Cincinnati area. In Figure 3-5, the intensity-duration curves are plotted on log-log paper with i_{av} as abscissas and t_d+b as ordinates. It is worth noting that for 1, 10 and 100-year frequencies, a and b are the same (53 and 10 respectively) and that for 5, 25 and 50-year frequencies, a and b also remain unchanged (85 and 15 respectively).

DEVELOPMENT OF SYNTHETIC STORM PATTERN

The mass of water occurring within any duration time can be obtained from the intensity-duration curve for a selected design frequency (equation 1).

The mass of water P (inches) precipitated is:

$$P = i_{av} \frac{t_d}{60} \qquad (2)$$

or

$$P = \frac{a}{(t_d+b)^c} \frac{t_d}{60} \qquad (3)$$

Initially, a completely advanced rainfall pattern can be developed that has the peak occurring at time zero, with no antecedent rainfall. The area under the curve of the hyetograph is:

Table 3-2. INTENSITY-DURATION CURVES OF THE FORM OF EQ.(1) FOR SELECTED DESIGN FREQUENCIES N. CINCINNATI AREA, OHIO.

Frequency N (years)	M* (in)	Av. intensity (in/hr)	a	b	c
1	1.10	1.10	53	10	0.910
2	1.35	1.35	44	10	0.820
5	1.70	1.70	85	15	0.900
10	1.95	1.95	53	10	0.770
25	2.25	2.25	85	15	0.825
50	2.50	2.50	85	15	0.810
100	2.75	2.75	53	10	0.680

*1-hour rainfall, in inches, to be expected once in N years obtained from isopluvial maps for Cincinnati.

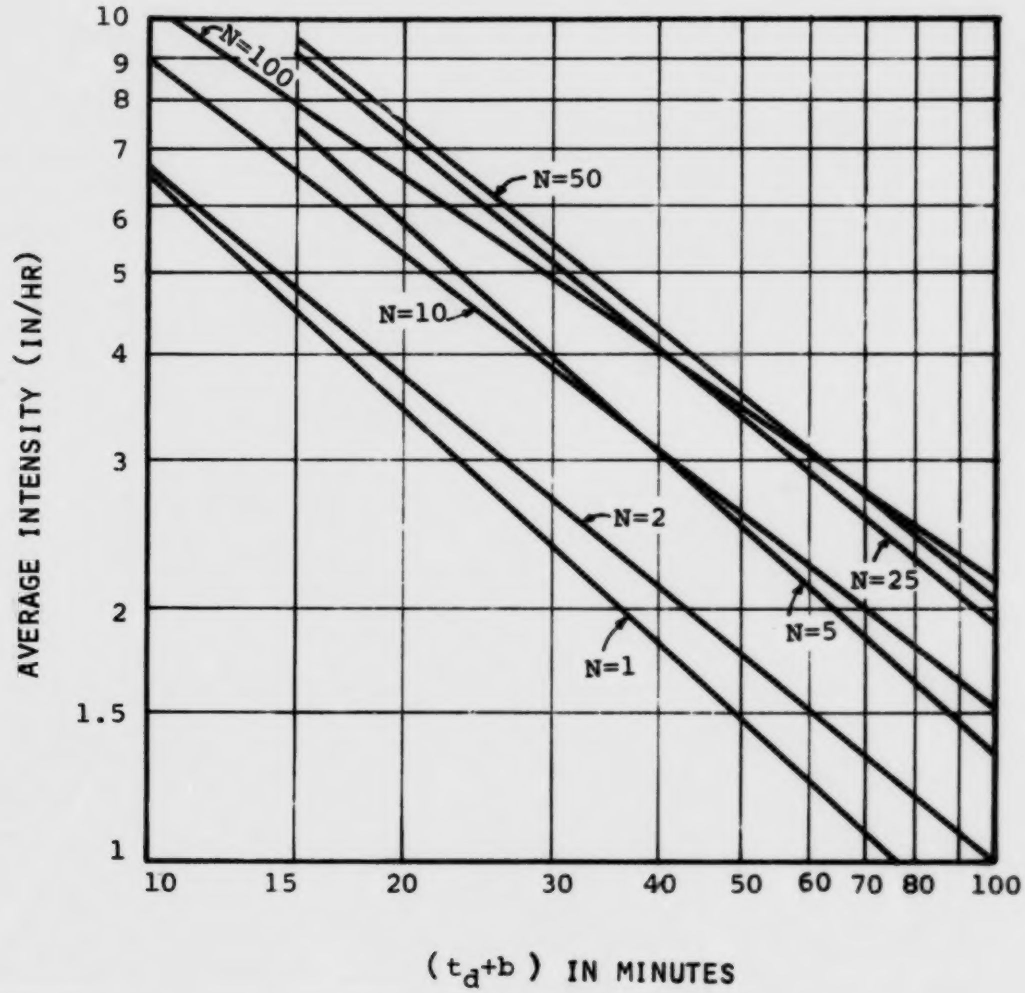

Figure 3-5. INTENSITY-DURATION CURVES OF THE FORM OF EQUATION (1), FOR SELECTED DESIGN FREQUENCIES N. CINCINNATI AREA, OHIO.

$$P = \frac{1}{60}\int_0^{t_d} i \, dt_d \qquad (4)$$

where i = ordinate of hyetograph (rainfall intensity in inches/hour)

Differentiating equation (4) with respect to t_d:

$$\frac{dP}{dt_d} = \frac{i}{60} \qquad (5)$$

Differentiating equation (3) with respect to t_d:

$$\frac{dP}{dt_d} = \frac{a}{60}\left[\frac{(t_d+b)^c - t_d c(t_d+b)^{c-1}}{(t_d+b)^{2c}}\right] = \frac{a}{60}\left[\frac{(1-c)t_d+b}{(t_d+b)^{1+c}}\right] \qquad (6)$$

Equating equations (5) and (6)

$$i = \frac{a[(1-c)t_d+b]}{(t_d+b)^{1+c}} \qquad (7)$$

Equation (7) represents a completely advanced rainfall pattern (peak occurring at time zero) that has the same average intensities as given by the rate duration curve. However, this equation must be modified to apply to any intermediate type storm pattern (a storm with peak after time zero). Within the maximum period of any rainfall, duration t_d can be separated into the period $t_b = rt_d$ which occurs before the peak measured from the peak to the left and the period $t_a = (1-r)t_d$ which occurs after the peak and is measured from the peak to the right. Obviously $t_d = t_a + t_b$. For r = 0, the storm is of the completely advanced type. For $0 < r < 1$ the storm pattern is of the intermediate type (Figure 3-6).

Substituting into equation (7), $t_d = t_b/r$:

Before the peak $$i_b = \frac{a\left[(1-c)\frac{t_b}{r} + b\right]}{\left[\frac{t_b}{r} + b\right]^{1+c}} \qquad (8)$$

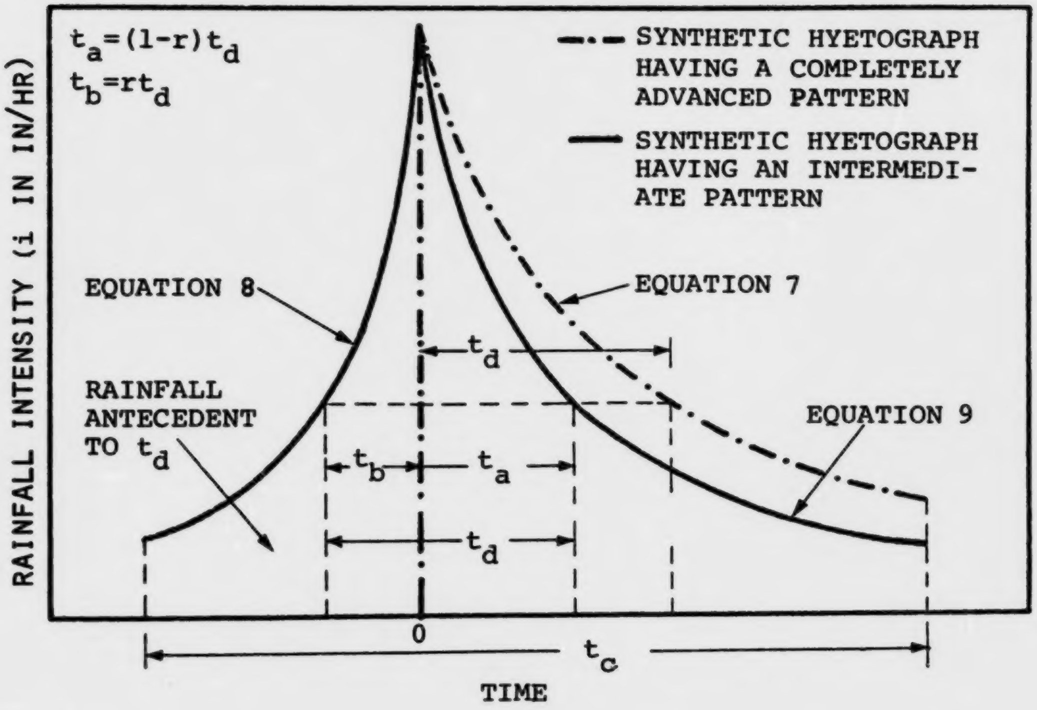

Figure 3-6 COMPLETELY ADVANCED AND INTERMEDIATE STORM PATTERNS.

Substituting into equation (7), $t_d = t_a/(1-r)$:

After the peak

$$i_a = \frac{a\left[(1-c)\ \frac{t_a}{1-r} + b\right]}{\left[\frac{t_a}{1-r} + b\right]^{1+c}} \qquad (9)$$

For $t_a = 0$ and $t_b = 0$, the magnitude of the peak is obtained from $a/(b^c)$. The synthetic hyetograph which results from equations (8) and (9) (Figure 3-6) has the same average intensity for all times of concentration as the intensity-duration curve (for a certain design frequency) from which the constants a, b and c were derived (Table 3-2).

The only unknown quantity in equations (8) and (9) is r. Therefore, it remains to assign the best value to the constant r for Cincinnati from the available rainfall records.

DETERMINATION OF r FROM ANTECEDENT RAINFALL

The mass precipitated (P_b in inches) before the peak within a duration t_d is:

$$P_b = rP = \frac{ra\ t_d}{60(t_d+b)^c} \qquad (10)$$

To assure adequate design, the maximum runoff that may occur with the design frequency must be determined. Therefore, the maximum (peak) runoff will occur when all parts of the watershed are contributing to the flow. The time required for this to happen is called time of concentration (t_c). A storm producing maximum runoff must be of maximum intensity and last long enough for all parts of the watershed to contribute to the flow concurrently. A storm of this type should necessarily have a duration equal to the time of concentration. Therefore, the total duration of the synthetic storm pattern is set equal to the longest time of concentration which for the Bloody Run watershed in Cincinnati is estimated to be 100 minutes. Since this duration is relatively long for an urban area, it is assumed that any small amount of rainfall preceding this period would produce little effect on runoff, therefore no antecedent rainfall is assumed prior to this 100-minute-long synthetic storm pattern.

The amount of antecedent rainfall (A in inches) preceding the duration t_d, can now be expressed as:

$$A = [P_b]_{t_d=t_c} - [P_b]_{t_d=t_d} = \frac{rat_c}{60(t_c+b)^c} - \frac{rat_d}{60(t_d+b)^c} \quad (11)$$

For a certain duration time t_d, the ratio A/r can be obtained from equation (11). Therefore, in order to find r, the mass A of the antecedent rainfall preceding t_d should be estimated first. Periods of 15, 30 and 60 minutes have been selected as duration times and Table 3-3 is a tabulation of the antecedent mass precipitated prior to each duration. Only those rainfalls were recorded which were considered excessive at least for some definite duration. The precipitation records were obtained from three raingages which recorded automatically the accumulative mass curve of each rainfall. The three raingages triangulate the Bloody Run Sewer Watershed in Cincinnati, Ohio and are named after their locations (Laidlaw Avenue, Ridge Road and Woodward School). Since the area of the Bloody Run Watershed is 2380 acres (3.7 square miles) the ratio of average precipitation to maximum point precipitation was considered equal to one [6]. Only two years of detailed rainfall records were available; however, this is considered to be sufficient since r is independent of frequency.

From Table 3-3 a statistical mean value of A is obtained for each duration time. These values are used to solve equation (11) and to obtain the corresponding values of r for the three durations under consideration. Through all durations r is to be constant, but the antecedent rainfall for the shorter durations is greater and of greater importance in affecting runoff than that for the longer durations. Therefore, the values of r obtained from different durations should be weighted in proportion to A in obtaining the average r value. This procedure is presented in Table 3-4.

The value of r obtained for the Cincinnati area example is 13/40. For Chicago, r = 15/40 [3] and for Gauhati, India, r was found to be 16/40 [1]. The mass antecedent rainfall for Cincinnati is plotted in Figure 3-7 using equation (11) for 1-year frequency. The statistical mean values of the mass antecedent rainfall of Table 3-3 are also plotted in Figure 3-7 for comparison.

Table 3-3. ANTECEDENT PRECIPITATION RECORDS FROM THREE STATIONS IN THE VICINITY OF BLOODY RUN WATERSHED, CINCINNATI, OHIO.

Date	Station*	Antecedent Mass A(in) Prior To the Maximum :		
		15 min. duration	30 min. duration	60 min. duration
5/12/70	L	0.040	0.040	0.040
	R	0.040	0.040	0.040
9/13/70	L	0.075	0.045	0
	W	0.125	0.042	0
9/25/70	W	0.290	0.290	0.290
9/26/70	W	0.120	0.080	0
10/9-10/70	L	0.060	0.030	0
	W	0.090	0	0
11/19-20/70	L	0.020	0.020	0.020
	R	0.095	0.095	0.080
12/11-12/70	L	0.067	0.047	0.012
12/21-22/70	R	0.067	0.054	0
2/4/71	L	0.550	0.500	0.400
	R	0.501	0.448	0.343
	W	0.220	0.150	0.050
2/19-20/71	R	0.050	0.010	0.005
3/19/71	L	0	0	0
4/27/71	L	0.012	0	0
	R	0.017	0	0
5/5-6/71	L	0.050	0.040	0
	W	0.065	0	0
	R	0.080	0.010	0
5/7/71	L	0.135	0.100	0.053
	W	0.670	0.670	0.660
5/19/71	R	0	0	0
6/7/71	R	0.095	0.070	0
6/14/71	L	0.060	0.030	0
6/19/71	W	0	0	0
7/10/71	R	0.200	0	0
7/14/71	R	0.025	0	0
	L	0.025	0	0
8/21-22/71	W	0.100	0	0
8/25-26/71	W	0.060	0	0
9/2/71	R	0.400	0.400	0.127
9/2-3/71	L	0.570	0.570	0.350
9/3-4/71	L	0.510	0.460	0.250
9/2-4/71	R	0.827	0.727	0.480
9/6/71	R	0	0	0
9/11-12/71	W	0.190	0.120	0

Table 3-3 (continued).

9/13/71	W	0.180	0.110	0
9/20/71	L	0.100	0.030	0
	R	0.250	0.050	0
	W	0.200	0.100	0.020
9/26/71	L	0.030	0	0
	R	0.635	0.535	0.160
	W	0.095	0.095	0
10/16/71	L	0.025	0	0
	W	0.070	0	0
11/2/71	R	0.010	0.010	0
11/19/71	R	0.050	0	0
11/24/71	W	0.100	0	0
12/15/71	L	0.040	0	0
	W	0.065	0	0
12/30/71	W	0.105	0.205	0.105
1/24/72	W	0	0	0
3/2/72	W	0	0	0
4/5/72	L	0.100	0.050	0
4/7/72	R	0.050	0.050	0
4/12/72	W	0	0	0
4/13/72	W	0.030	0	0
4/16/72	W	0.190	0.100	0
5/8/72	L	0.200	0.150	0
5/9/72	W	0.290	0.220	0.010
5/29/72	L	0	0	0
	R	0.300	0	0
5/30/72	L	0.100	0	0
	R	0	0	0
6/15/72	L	0	0	0
6/21/72	W	0.130	0.080	0
	R	0	0	0
7/20/72	L	0.360	0.340	0.220
Number of excessive station rainfalls		71		
Mean A (inches)		0.1437	0.1016	0.0523

*L, R and W represent the Longview, Ridge and Woodward raingage stations in the vicinity of the Bloody Run Sewer Watershed, Cincinnati, Ohio.

Table 3-4. DETERMINATION OF "r" FROM MASS ANTECEDENT RAINFALL FOR A 1-YEAR DESIGN FREQUENCY, BLOODY RUN WATERSHED, CINCINNATI, OHIO.

t_d	t_d+10	$(t_d+10)^{0.91}$	$\frac{53\ t_d}{60(t_d+10)^{0.91}}$	$\frac{A}{r}$*	mean A**	r	Ar
15	25	18.7	0.710	0.517	0.1437	0.278	0.0399
30	40	28.6	0.927	0.300	0.1016	0.339	0.0344
60	70	48.0	1.104	0.123	0.0523	0.425	0.0222
					$\Sigma A = 0.2976$		$\Sigma(Ar) = 0.0965$
Weighted value of r = 0.325 = 13/40							

*From the 1-yr frequency equation $A = \frac{r}{60}\left[\frac{53 \times 100}{(100+10)^{0.91}} - \frac{53\ t_d}{(t_d+10)^{0.91}}\right]$

**From Table 3-3

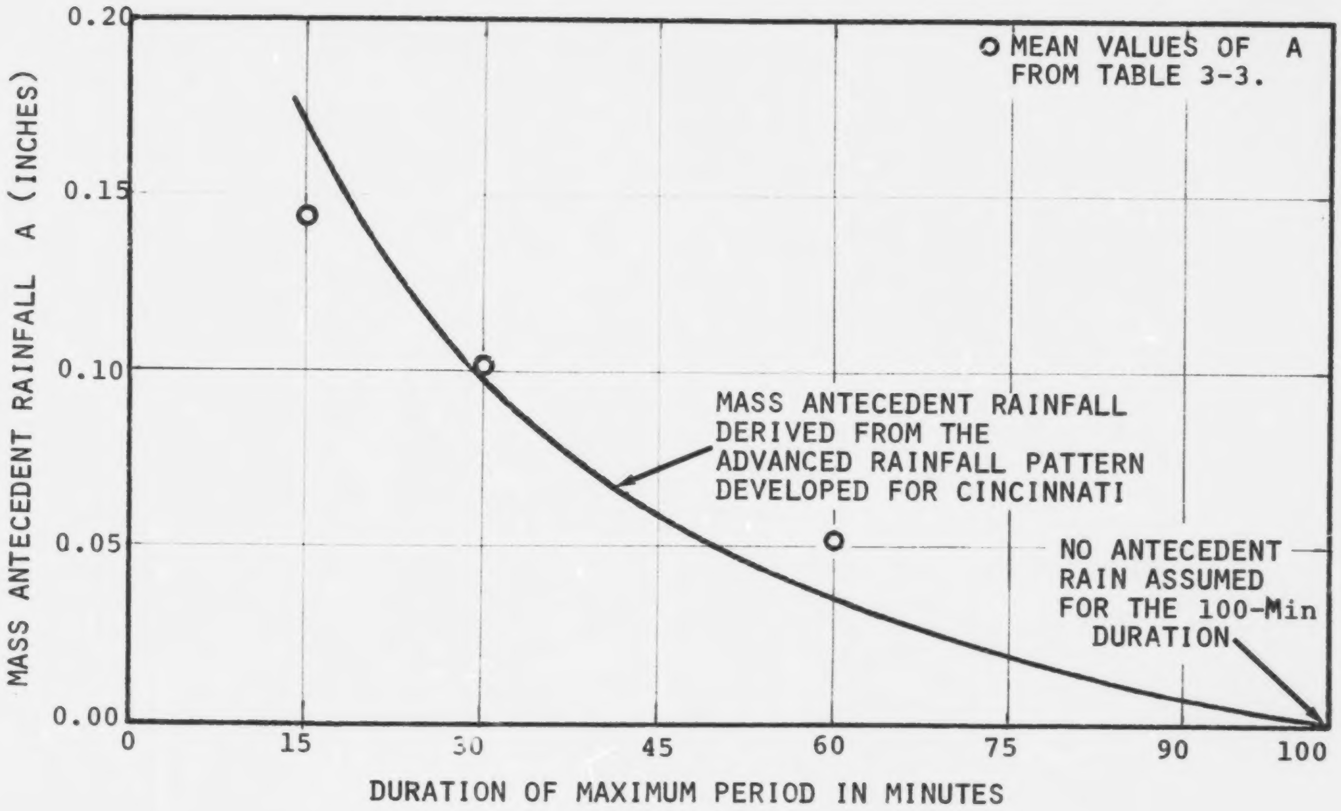

Figure 3-7. COMPARISON OF THE MASS ANTECEDENT RAINFALL OCCURRING IN THE 1-YEAR SYNTHETIC STORM PATTERN WITH THAT OF THE MEAN VALUES OF ACTUAL RAINFALLS.

Table 3-5. TABULATIONS OF THE SYNTHETIC STORM PATTERN EQUATIONS FOR SELECTED DESIGN FREQUENCIES. CINCINNATI, OHIO (EQUATIONS 8 & 9).

Frequency	i_b(in/hr) before the peak*	i_a(in/hr) after the peak*	peak (in/hr)
1	$\frac{14.7\ t_b + 530}{(3.08\ t_b + 10)^{1.91}}$	$\frac{7.1\ t_a + 530}{(1.48\ t_a + 10)^{1.91}}$	6.55
2	$\frac{24.4\ t_b + 440}{(3.08\ t_b + 10)^{1.82}}$	$\frac{11.7\ t_a + 440}{(1.48\ t_a + 10)^{1.82}}$	6.67
5	$\frac{26.2\ t_b + 1275}{(3.08\ t_b + 15)^{1.90}}$	$\frac{12.6\ t_a + 1275}{(1.48\ t_a + 15)^{1.90}}$	7.42
10	$\frac{37.5\ t_b + 530}{(3.08\ t_b + 10)^{1.77}}$	$\frac{18.1\ t_a + 530}{(1.48\ t_a + 10)^{1.77}}$	9.00
25	$\frac{46.8\ t_b + 1275}{(3.08\ t_b + 15)^{1.825}}$	$\frac{22.1\ t_a + 1275}{(1.48\ t_a + 15)^{1.825}}$	9.15
50	$\frac{52.4\ t_b + 1275}{(3.08\ t_b + 15)^{1.81}}$	$\frac{25.2\ t_a + 1275}{(1.48\ t_a + 15)^{1.81}}$	9.50
100	$\frac{52.2\ t_b + 530}{(3.08\ t_b + 10)^{1.68}}$	$\frac{25.2\ t_a + 530}{(1.48\ t_a + 10)^{1.68}}$	11.00

*t_a and t_b are zero at the peak (Figure 3-6).

Table 3-6. TIME-INTENSITY PATTERN OF 1-YEAR FREQUENCY (No. 1.1 CURVE) FOR CINCINNATI, OHIO

Time t_d (min)	Average Intensity i_{av} (in/hr)	Accumulated Precip. (in) $i_{av}t_d/60$	Δt (min)	ΔP (in)	Precip. i (in/hr) during increment 60 $\Delta P/\Delta t$
0		0			
			5	0.375	4.50
5	4.50	0.375			
			5	0.207	2.48
10	3.50	0.582			
			5	0.118	1.42
15	2.80	0.700			
			5	0.100	1.20
20	2.40	0.800			
			5	0.075	0.90
25	2.10	0.875			
			5	0.050	0.60
30	1.85	0.925			
			10	0.075	0.45
40	1.50	1.000			
			10	0.060	0.36
50	1.27	1.060			
			10	0.040	0.24
60	1.10	1.100			
			10	0.040	0.24
70	0.98	1.140			
			10	0.020	0.12
80	0.87	1.160			
			10	0.040	0.24
90	0.80	1.200			
			10	0.030	0.18
100	0.70	1.170			

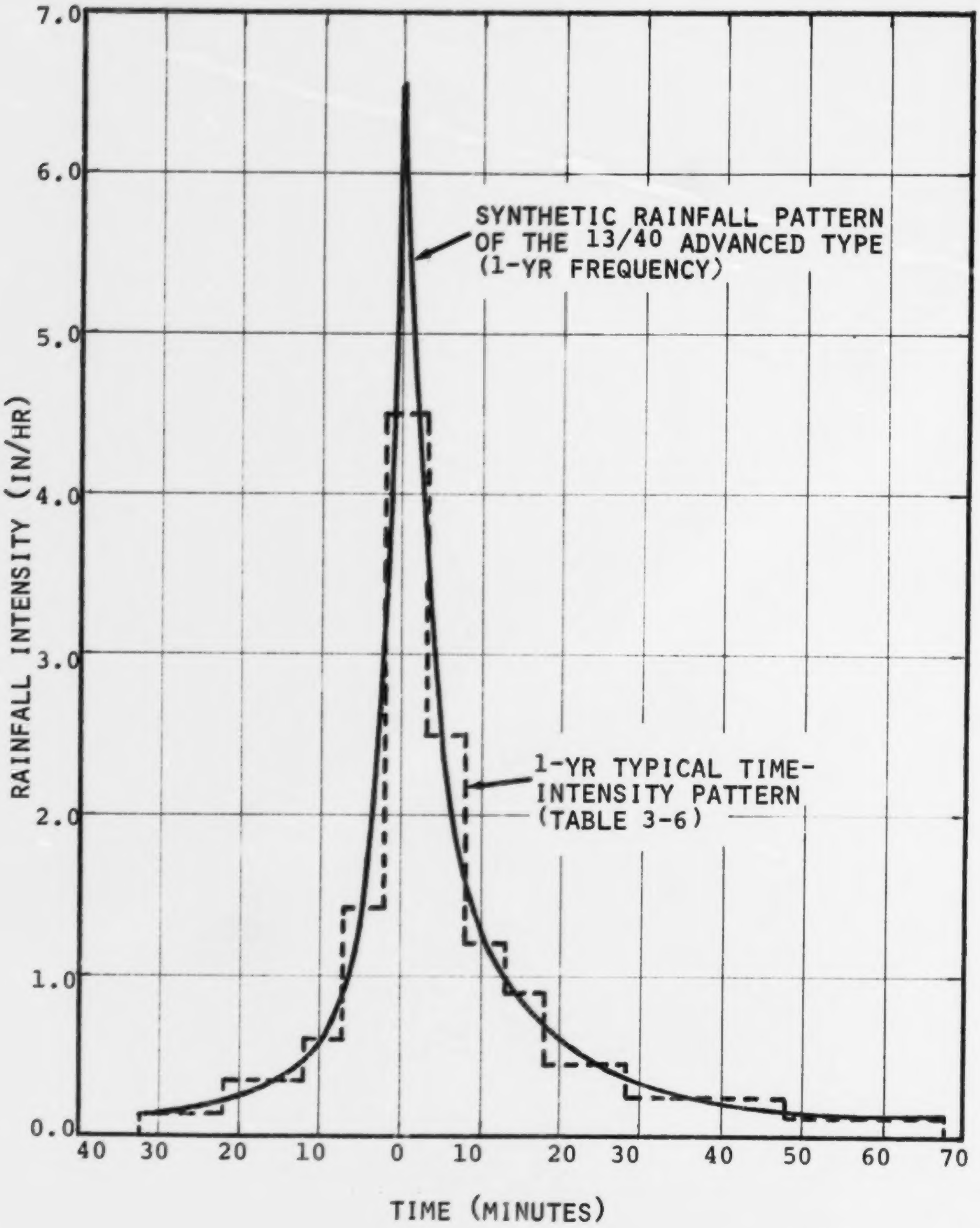

Figure 3-8. COMPARISON OF THE 1-YEAR SYNTHETIC RAINFALL PATTERN AND THE 1-YR TYPICAL TIME-INTENSITY PATTERN FOR THE CINCINNATI AREA, OHIO.

Equation (8) generates the rising limb of the intermediate storm pattern and equation (9) generates the recession of the hyetograph. These two equations are tabulated in Table 3-5 for seven different design frequencies after substituting a, b, c and r with their values. A reasonable approximation is to consider r not varying for different frequencies [3].

Another method may be used [6] to develop an arbitrarily arranged time-intensity pattern for a 1-year rain at Cincinnati, Ohio. Since a typical intensity-duration curve can be derived from a single intense rain, an intensity-duration curve may be used to determine a time-intensity pattern for a rain of any selected frequency. A 1-year rainfall at Cincinnati falls on the curve No. 1.1 of Figure 3-3. From this curve it can be seen that the probable average intensity during the maximum 60 minutes of a 1-year rain would be 1.10 inches per hour, for the maximum 90 minutes, 0.80 inches per hour, and so on. The maximum 60 minutes of rain include the maximum 30 min. of rain as well as the second largest 30-min. period. The computations for the derivation of the intensities are shown in Table 3-6. The arrangement of the derived time-intensity pattern is arbitrary. Therefore, the intensities obtained can appear in any order. In Figure 3-8 this arbitrary time-intensity pattern is plotted in such a way as to best fit the corresponding 1-year synthetic rainfall pattern of the 13/40 advanced type. The agreement is remarkably good.

SUMMARY

Synthetic Storm Patterns for seven selected design frequencies have been derived from the corresponding intensity-duration curves developed for a Cincinnati, Ohio watershed and from the antecedent precipitation immediately preceding the maximum period of any durations obtained from rainfall records at three locations in the Cincinnati area.

Deterministic mathematical models simulating urban runoff resulting from a given rainfall can be used successfully in sewer system design and analysis only if the proper synthetic hyetograph pattern for the desired design frequency is used as input. Therefore, this consideration and selection must be made prior to using models such as the EPA Storm Water Management Model or the University of Cincinnati Urban Runoff Model for design purposes.

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Section 3

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SECTION 4

DETERMINATION OF INFILTRATION CAPACITY CURVES FOR USE IN MATHEMATICAL MODELS

SYNOPSIS

During recent years a number of deterministic mathematical models have been developed for the calculation of runoff hydrographs from urban watersheds. These models include the Chicago Method [14], the EPA Storm Water Management Model [9], and the University of Cincinnati Urban Runoff Model [10], and provide a powerful tool for the improvement of sewer system design. Most of the existing mathematical models use Horton's equation for the simulation of the infiltration process. The shape of the infiltration capacity curve, for a given soil, greatly depends on the antecedent precipitation. Therefore, it is important that Horton's constants be related with "dry" and "wet" antecedent conditions [3], [4].

The purpose here is to relate Horton's initial rate of infiltration and decay rate of infiltration with the antecedent precipitation. By obtaining a precipitation record for the area under consideration and by performing a few inexpensive flooding-type tube infiltrometer tests, Horton's infiltration capacity curve can be obtained for the soil for which tests were performed for any specified antecedent conditions. A case study is presented to illustrate the application of the method.

ALGEBRAIC INFILTRATION EQUATIONS

Infiltration is the process during which water enters the surface strata of the soil and moves toward the water table. The maximum rate at which a soil in any given condition is capable of absorbing water is called its infiltration capacity. It has generally been found that during continuous rain the infiltration capacity of a given terrain decreases, at first rapidly, then approaches an asymptotic line to a constant value.

Empirical attempts to fit experimental data have resulted in many algebraic infiltration equations, the most well known of which are Kostiakov's equation (1932), Horton's equation (1940), Philip's equation (1954) and Holtan's equation (1961). These are reviewed in the following.

a) The Kostiakov Equation

Kostiakov [5] proposed an empirical equation of the form:

$$f = cnt^{n-1} \tag{1}$$

f = infiltration capacity rate in inches per hour

t = time elapsed since water started being applied, in minutes

c,n = coefficients

Experiments with a recent sedimentary soil classified as Caldwell loam conducted by Lewis [6] in 1937 showed the exponential factor n to vary from 0.24 to 0.7 and the coefficient c from 1.86 to 9.60.

The volume of water M (inches) infiltrated in a time t, is

$$M = \int_0^t \frac{f}{60}\, dt = \frac{c}{60}\, t^n \tag{2}$$

The actual value of n obtained when Kostiakov's equation is fitted to data will clearly depend on the range of t. Also the physical significance of c depends directly on n and therefore indirectly on the range of t. For a common time range, n will be closer to 0.5 for fine-textured and initially dry soils. For wet and sandy soils in which gravity becomes important more rapidly, n will be closer to 1.0. The simplicity of Kostiakov's equation has encouraged its use mainly in flood irrigation studies, where the usual procedure is to fit c and n from test data on the plot under consideration.

Since, for $n = 1.0$ and $t \rightarrow \infty$ the lim $f = 0$, it should be expected that equation (1) becomes less accurate as t increases. This limitation can be avoided by considering that equation (1) holds for $f > f_c$ (where f_c = constant infiltration capacity in inches per hour) and that for:

$$t > (cn/f_c)^{1/(1-n)} \longrightarrow f = f_c$$

Kostiakov's equation is incapable of providing a value for the initial infiltration capacity f_o since for $t \rightarrow 0$, $\lim f = \infty$.

Kostiakov's equation written in logarithmic form becomes:

$$\log f = \log (cn) + (n-1) \log t \qquad (3)$$

In this form the equation is that of a straight line in which log f and log t are the variables. The slope of the line is (n-1).

b) The Horton Equation

An exhaustion process is one in which the rate of performing work is proportional to the amount of work remaining to be performed. Such processes are common in nature and follow an inverse exponential law. Horton [1] assumed that the reduction of the infiltration capacity as rain continues is an exhaustion type process and therefore expressed the infiltration capacity as a function of the time elapsed from the beginning of the rain.

During infiltration into a soil, the rate of change of the infiltration capacity df/dt (negative because f is decreasing) can be assumed proportional to the difference between the present infiltration capacity f and the ultimate constant infiltration capacity f_c. Introducing a positive factor of proportionality k, the following results:

$$-\frac{df}{dt} = k(f-f_c)$$

or

$$d\ln(f-f_c) = -kdt$$

$$\ln(f-f_c) = -kt + C$$

when $t = 0$ then $f = f_o$ and $C = \ln(f_o-f_c)$

therefore

$$\ln \frac{f - f_c}{f_o - f_c} = -kt$$

or

$$f = f_c + (f_o-f_c)e^{-kt} \qquad (4)$$

Equation (4) is Horton's equation of infiltration capacity, where:

f = infiltration capacity at any time t (inches/hr)

t = time in minutes elapsed from the beginning of the infiltration capacity curve

f_c = the constant rate which f is approaching asymptotically as time continues

f_o = initial infiltration capacity (when $t = 0$)

k = decay rate of infiltration which is a positive constant with (1/minute) units.

Equation (4) applies only in cases where the rain starts and continues with an intensity i greater than f. Then time t in Horton's equation is also the time in minutes elapsed from the beginning of the rainfall. This condition limits considerably the application of Horton's equation without modification [10, 11] in real cases. However, for infiltration tests where adequate water is applied to enclosed sample areas, equation (4) is the case.

The area under the infiltration capacity curve represents the accumulated volume of water in inches infiltrated up to a time t, which is:

$$M = \int_0^t \frac{f}{60}\,dt = \frac{f_c t}{60} + \frac{(f_o - f_c)}{60\,k}\,(1 - e^{-kt}) \qquad (5)$$

Horton's equation written in logarithmic form becomes:

$$\log(f - f_c) = \log(f_o - f_c) - kt \log e \qquad (6)$$

In this form the equation is that of a straight line in which t and $\log(f-f_c)$ are the variables. The slope of the line is $(-0.4343k)$.

Advantage of Horton's equation is that for $t \to 0$, $\lim f = f_c = 0$ and disadvantage is the need for three parameters f_o, f_c, and k.

c) The Philip Equation

Philip [12,13] has solved the partial differential equation of soil moisture flow for a homogeneous soil with uniform initial moisture content and an excess water supply at the surface. Because of the rapid convergence of the solution which is in the form of an infinite series, only the first two terms were retained and a simple infiltration equation with physical significance evolved:

$$f = \frac{30\ S}{\sqrt{t}} + A \tag{7}$$

where f = infiltration capacity rate in inches/hour

S = a measure of the capillary uptake or removal of water called by Philip "sorptivity". Sorptivity is a property of the soil with some resemblance to permeability with units inches per $\text{min.}^{-1/2}$

A = constant with units inches per hour.

It can be noted that in Kostiakov's equation if $n = 0.5$ and $t \to 0$, then $c = 30S$.

Philip stated that A is not equal to f_c, for very large t, and that equation (7) should be modified semi-empirically for large t.

The volume of water in inches infiltrated in a time t is:

$$M = \int_0^t \frac{f}{60}\, dt = \frac{S\sqrt{t}}{2} + \frac{At}{60} \tag{8}$$

Equation (7) plots as a straight line with f and $1/\sqrt{t}$ as variables and slope 30S.

d) The Holtan Equation

Holtan [2] based on a substantial volume of field data defined the infiltration rate of a soil as a function of the exhaustion of soil moisture storage:

$$f = f_c + q(F-M)^m \tag{9}$$

in which f = infiltration capacity in inches per hour,

f_c = final steady infiltration capacity in inches per hour (same as in Horton's equation), F = initial available moisture storage (equal to the volumetric difference between pore saturation and the 15-bar or permanent wilting percentage for the soil zone under consideration),

M = mass of water infiltrated in inches,

q = constant for a particular soil in given condition, with value between 0.25 and 0.80,

m = constant dependent on the soil type.

Equation (9) relates infiltration capacity to the soil moisture level and is not directly time dependent. The value of m according to Philip is $m = -0.5$. When the mass infiltrated equals the moisture storage the infiltration rate is equal to f_c.

Holtan's equation in logarithmic form becomes:

$$\log(f-f_c) = \log q + m \log(F-M) \qquad (10)$$

In this form the equation is that of a straight line in which $\log(f-f_c)$ and $\log(F-M)$ are the variables. The slope of the line is m.

THE ANTECEDENT PRECIPITATION INDEX

Infiltration capacity is affected primarily by the type of soil, soil texture, the depth and thickness of its more permeable layers, and its prior history of land use. Coarser soils such as gravels and sands will have a lower decay rate of infiltration k and a higher constant rate of infiltration f_c than finer soils such as silts and clays. Temperature affects infiltration also. In general, a greater portion of rainfall becomes runoff during the cooler months of the year, for example during March and April in mid-west U.S.A.

For a given soil, the most important factor affecting infiltration is the moisture content of the soil which is related to the antecedent weather conditions. The antecedent precipitation index (API) reflects the soil moisture content on any given day as a function of the rainfall which fell prior to that day. Methods for tabulating this index include that developed by Horner (1943) for the Black

Lands of Texas, and a method developed by the U.S. Department of Agriculture. The method chosen here is currently used by the U.S. Weather Service; it is simple but also very inclusive. The value of the API decreases daily by some amount (10% was used in the present analysis) and is increased by the amount of the rainfall in inches on any given day. It is evident that a starting point is needed in calculating API. It is usually sufficient to go back a couple of months prior to the infiltration tests and to start generating API values. The first value of API is to be assumed arbitrarily, but after several weeks the API value obtained will be the same no matter what initial API value was selected. This concept is similar to that of a Markov process, where the n^{th} state is defined by the $(n-1)^{th}$ state. No matter what the initial state, enough changes of state will eventually result to the same final state.

EXPERIMENTAL SETUP

The experimental setup used for the infiltration tests consisted of a flooding-type tube infiltrometer. Artificial application of water to the enclosed sample areas was performed by two graduated burettes. The single tube infiltrometer was a hollow thin-walled metal cylinder sharpened on the bottom edge with inside diameter 20.9 cm, area equal to 344 sq.cm. and length equal to 41 cm.

To perform an infiltration test, the grass was cut to a maximum height of one inch (2.54 cm). The tube was then jacked into the ground at sufficient depth so as to prevent spreading of the infiltrating water during the experiment. Jacking was performed in such a way as to minimize disturbance of the soil structure. At time zero, 450 milliliters of water were poured into the cylinder, the equivalent of a head of water equal to 0.5 inches (1.27 cm). Immediately afterwards water was applied from the graduate burettes to maintain the head of water constant. Readings, taken at successive time intervals, of the volumes of water required to maintain the constant head, determine the rate with which volumes of water infiltrate into the ground. Thus the curve of the accumulated volume of water infiltrated is obtained versus time. A typical installation for infiltration tests is shown in Figures 4-1, 4-2.

By using flooding-type tube infiltrometers, a simple and direct method of determining the capacity at which the ground is capable of absorbing water is obtained, but the effect of the raindrop impact on the ground surface is absent. In reality, increased raindrop size and rainfall intensity cause the infiltration capacity to drop off more rapidly. Also the effect of compression of the entrapped air is absent because of free lateral escape of air at the base of the tube. The results of the infiltration tests are also affected by the unavoidable disturbance of the soil structure due to the jacking of the tube into the ground. However, in spite of the above disadvantages, it was decided that due to their simplicity, flooding-type tube infiltrometer tests should be preferred since more laborious techniques such as rain simulators, ring-type and sprinkling-type infiltrometers present the same disadvantages, although to a lesser degree [7].

CASE STUDY

A combined sewer watershed of diversified usage in the City of Cincinnati, Ohio, was selected in 1969 for the purpose of collecting data to develop and verify storm water management models for urban areas. The watershed comprises 2,380 acres of rolling terrain and is referred to as the Bloody Run Sewer Watershed.

The infiltration tests were performed on the Longview Hospital grounds which are located at the west part of the Bloody Run Sewer Watershed. The soils there have been classified by the Soil Conservation Service as silty clay loams, underlain by clay loams. However, separate tests performed at the University of Cincinnati indicated a soil classification (based on the Unified Soil Classification System) of ML - OL, well-graded sandy, clayey silt.

At the test site the grass was cut to a maximum height of 2.5 cm. The infiltrometer tube was jacked into the soil to a depth of 35 cm, about 15 cm of which was the top soil underlain by clay loam. At time zero a volume of 450 milliliters of water, equivalent to 0.5 inches of head, was poured into the cylinder. Immediately after, water was titrated from the graduated burettes in such a way as to maintain the head of 0.5 inches constant. During each infiltration test, while maintaining the constant head, readings of the graduated burettes at successive time intervals of one or two minutes provided the accumulated volume of water infiltrated into the ground versus time (infiltration mass curve). The data collected are presented in Table 4-1.and additional information such as time of day, temperature and topsoil thickness, can be found in Table 4-2.

Considering the infiltration capacity to remain constant over each time increment Δt, a stepwise infiltration capacity curve can be obtained by writing:

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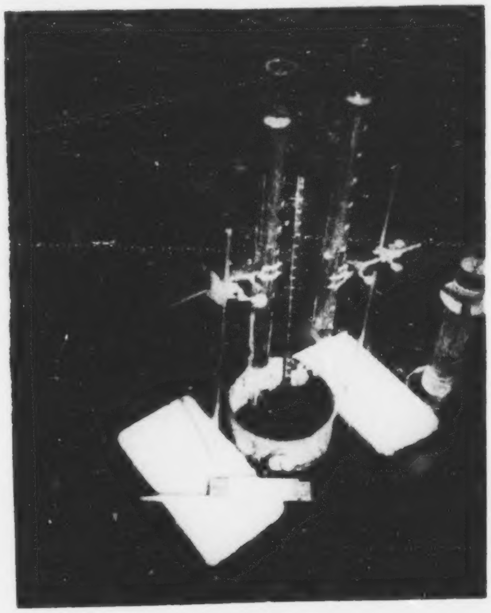

Figure 4-1

FLOODING TYPE TUBE INFILTROMETER

EXPERIMENTAL SETUP

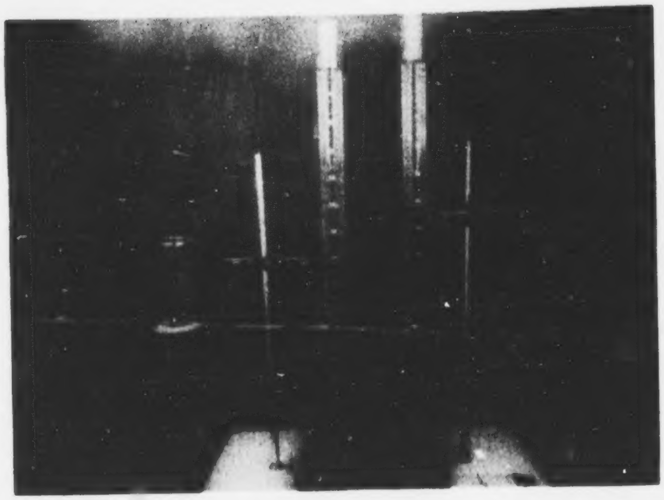

Figure 4-2

FLOODING TYPE TUBE INFILTROMETER

EXPERIMENTAL SETUP

$$f_{i+1/2} = \frac{23.6(V_{i+1}-V_i)}{A_r \cdot \Delta t} \qquad \text{with } i = 1,2,\ldots N \tag{11}$$

where $f_{i+1/2}$ = infiltration capacity in inches per hour, constant over the time interval from $(i\Delta t)$ to $(i+1)\Delta t$.

V_i = accumulated volume of water infiltrated into the ground, in milliliters, until a time $(i\,\Delta t)$ from the beginning of the test

Δt = interval between observations in minutes, not necessarily constant during the whole test

A_r = area of tube infiltrometer (sq. cm.)

N = total number of observations made during the infiltration test

Each infiltration test was considered to be completed when the infiltration mass curve became a straight line, i.e. when the rate with which water infiltrated into the ground remained constant.

Horton's initial infiltration capacity, f_o, and the decay rate of infiltration, k, depend on soil type and initial moisture content. The final infiltration capacity, f_c, theoretically should be constant for a particular location. The present study revealed small variations in the values of f_c from one storm to another (Table 4-2) and the use of the average of all observed values is suggested as a good approximation for f_c. Better fits to experimental data by using varying f_c values led Mein and Larson [8] to state that the good fit with experimental data by using Horton's equation might be due to the existence of three parameters in the equation. Herein only the two parameters of Horton's equation, f_o and k, were fitted to the experimental results by a linear regression analysis and Horton's equation exhibited better fit than Kostiakov's, Philip's and Holtan's equations. Also, since it seems that Civil Engineers tend to use Horton's equation with more confidence, it was decided to work throughout the following with Horton's equation.

The method of least squares was used to determine the best-fitting straight line for Horton's equation (6) considering that the principal errors were present only in the values of $\log(f-f_c)$. The values obtained for the

Table 4-1. RESULTS OF THE INFILTRATION TESTS, LONGVIEW, BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO 1972.

Time (min)	Accumulated Volume of Water Infiltrated (ml)										
	5/6	5/17	6/1	6/2	6/5	6/13	6/27	6/28A	6/28B	6/30	7/2
0	0	0	0	0	0	0	0	0	0	0	0
1	540	600	335	415	375	345	400	280	650	380	325
2	850	818	425	650	685	690	710	500	870	680	635
3	1200	933	475	733	915	950	910	685	1070	1020	850
4	1305	995	502	778	1033	1090	1060	885	1270	1220	1025
5	1415	1032	521	813	1125	1153	1160	1070	1370	1338	1175
6	1500	1059	535	837	1175	1205	1235	1165	1435	1433	1300
8	1552	1082	550	849	1195	1242	1275	1205	1485	1513	1390
10	1589	1099	561	861	1214	1257	1310	1240	1520	1553	1480
12	1622	1109	571	870	1228	1272	1342	1285	1560	1608	1570
14	1642	1119	581	878	1239	1292	1373	1330	1595	1654	1655
16	1658	1127	589	895	1248	1312	1403	1368	1629	1702	1735
18	1673	1136	596	901	1255	1326	1431	1403	1670	1777	1810
20	1687	1144	603	907	1262	1336	1458	1438	1704	1832	1870
22	1700	1151	609	913	1269	1348	1483	1478	1736	1892	1920
24	1712	1158	615	919	1276	1362	1505	1509	1765	1936	1970
26	1723	1164	621	925	1282	1374	1530	1539	1795	1986	2010
28	1738	1170	627	931	1288	1385	1552	1568	1821	2030	2045
30	1751	1176	633	937	1294	1394	1572	1593	1849	2063	2070
32	1762	1182				1404	1592	1616	1874	2103	2100
34	1775	1188				1414	1612	1637	1898	2143	2128
36	1786	1194				1424	1620	1654	1916	2170	2168
38	1796	1200				1433	1637	1670	1936	2199	2193
40	1806	1206				1442	1655	1690	1951	2221	2220
42	1815					1450	1672	1705	1965	2240	2242
44	1823					1458	1692	1719	1976	2259	2267
46	1831						1709	1731	1986	2275	2288
48	1839						1724	1745	1995	2290	2307
50	1847						1736	1757	2005	2302	2324
52	1855						1746	1767	2014	2313	2339
54	1863						1755	1776	2024	2324	2354
56	1871						1765	1787	2033	2334	2366
58	1879						1774	1795	2041	2344	2376
60	1887						1783	1804	2049	2354	2385
62							1791	1813	2057	2364	2394
64							1799	1821	2065	2374	2402
66							1807	1828		2384	2410
68							1815				2418
70							1823				2426

initial infiltration capacities f_o, the decay rates of infiltration k and the linear correlation coefficients r are tabulated in Table 4-2 for each infiltration test. The computed linear correlation coefficient for all the tests was very close to -1. If all points lie on the regression line, then $r = \pm 1$. If $r = 0$, the regression does not explain anything about the variation of $\log (f-f_o)$, and the regression line is horizontal. Therefore, Horton's equation fitted quite well the experimental data.

For the calculation of the API values which are listed in Table 4-3, the value 0.30 was selected as a starting API on March 14 (about seven weeks prior to the infiltration tests), since that was after a prolonged dry period. If 0.60 or 0.00 was chosen as a starting API, the index six weeks later would converge on the same values that were obtained when starting with an index of 0.30. The daily rainfall was obtained from a raingage located at the Longview test site which recorded automatically the accumulative mass curve of each rainfall. However, this was not necessary since the same rainfall data could be obtained from the U.S. Weather Bureau records for that area.

The Antecedent Precipitation Index is plotted versus the initial infiltration capacity f_o and the decay rate of infiltration k in Figure 4-3. For a different kind of soil these curves should be expected to be different. When the API is high, the moisture content of the soil is high and f_o has low values. As the API lowers, the moisture content of the soil correspondingly lowers, resulting in higher values of f_o. However in the present case study at the Longview grounds, a sudden decrease of f_o is noted for low API values. This occurred because of clay admixtures present in the top soil, which subjected to wetting after a prolonged dry period (low API) swelled and reduced the infiltration capacity during the initial period of the tests. Also, when the Longview soil becomes very dry, the surface contains many fine particles which when infiltration begins are deposited in the interstitial spaces, thus reducing the infiltration capacity. The same reasoning applies for the relationship between API and the decay rate of infiltration. Figure 4-4 presents a plot of API versus the accumulated mass infiltrated (in inches) during each test as obtained from equation (5) by using the corresponding values of f_o, f_c and k. Values of M are listed in Table 4-2. As would be expected, for smaller API values the accumulated mass infiltrated becomes bigger since the

Table 4-2. SUMMARY OF THE RESULTS OF THE INFILTRATION TESTS. LONGVIEW, BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Date	Time	Temp. °F	Topsoil thickness in. (cm)	API	Horton's Constants			Linear Correlation Coeff. r	Mass infiltrated M in inches (cm) in 60 minutes
					f_o in. (cm)	f_c in. (cm)	k (1/min)		
5/6/72	1420	75	7.5(19.1)	0.95	6.78(17.22)	0.27(0.69)	0.12	-0.83	1.17(2.97)
5/17/72	1700	78	7.0(17.8)	2.26	7.21(18.31)	0.21(0.53)	0.22	-0.81	0.74(1.88)
6/1/72	1700	70	7.0(17.8)	2.34	4.07(10.34)	0.21(0.53)	0.22	-0.77	0.50(1.27)
6/2/72	1500	75	7.0(17.8)	2.11	9.89(25.12)	0.21(0.53)	0.32	-0.80	0.71(1.80)
6/5/72	1350	85	8.5(21.6)	1.54	12.54(31.85)	0.21(0.53)	0.25	-0.84	1.03(2.62)
6/13/72	1340	75	9.0(22.9)	0.84	5.12(13.00)	0.27(0.69)	0.13	-0.80	0.89(2.26)
6/27/72	1900	87	10.0(25.4)	0.85	5.46(13.87)	0.27(0.69)	0.08	-0.85	1.34(3.40)
A6/28/72	1045	76	9.0(22.9)	0.76	6.36(16.15)	0.24(0.61)	0.07	-0.89	1.67(4.24)
B6/28/72	1210	76	10.0(25.4)	0.76	7.50(19.05)	0.27(0.69)	0.09	-0.88	1.60(4.06)
6/30/72	1325	70	11.0(27.9)	0.96	10.26(26.06)	0.34(0.86)	0.09	-0.89	2.15(5.46)
7/2/72	1330	81	11.0(27.9)	0.77	10.35(26.29)	0.27(0.69)	0.08	-0.92	2.35(5.97)

Table 4-3. THE API FOR LONGVIEW, BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO, 1972.

Date	Rain (in)	API	Date	Rain (in)	API	Date	Rain (in)	API	Date	Rain (in)	API
3/14		0.30	4/12	0.20	1.71	5/11		2.74	6/9		1.27
15	0.25	0.52	13	0.70	2.24	12		2.47	10		1.14
16	0.80	1.27	14		2.02	13	1.20	3.45	11		1.03
17		1.14	15	0.35	2.17	14		3.10	12		0.93
18	0.10	1.13	16	1.05	3.00	15		2.79	13	0.25	1.09
19		1.02	17		2.70	16		2.51	14		0.98
20		0.92	18		2.43	17		2.26	15	1.20	2.08
21	1.00	1.83	19	0.60	2.79	18		2.03	16		1.87
22	0.10	1.75	20	0.25	2.76	19		1.83	17		1.68
23		1.58	21		2.48	20		1.65	18		1.51
24		1.42	22	0.45	2.68	21		1.48	19		1.36
25		1.28	23		2.41	22		1.33	20		1.22
26		1.15	24		2.17	23		1.20	21	0.50	1.60
27		1.03	25		1.95	24		1.08	22		1.44
28		0.93	26		1.75	25		0.97	23		1.29
29	0.20	1.04	27		1.57	26		0.87	24		1.16
30		0.94	28		1.41	27		0.78	25		1.04
31		0.85	29		1.27	28		0.70	26		0.94
4/1		0.76	30		1.14	29	0.80	1.43	27		0.85
2		0.68	5/1	0.35	1.38	30	1.60	2.89	28	0.15	0.91
3	0.85	1.46	2	0.10	1.34	31		2.60	29	0.25	1.07
4		1.31	3	0.10	1.31	6/1		2.34	30		0.96
5		1.18	4		1.18	2		2.11	7/1		0.86
6		1.06	5		1.06	3		1.90	2		0.77
7		0.95	6		0.95	4		1.71	3		0.69
8		0.85	7		0.85	5		1.54	4	0.50	0.67
9	1.10	1.86	8	3.00	3.76	6	0.35	1.74	5		0.60
10	0.20	1.87	9		3.38	7		1.57			
11		1.68	10		3.04	8		1.41			

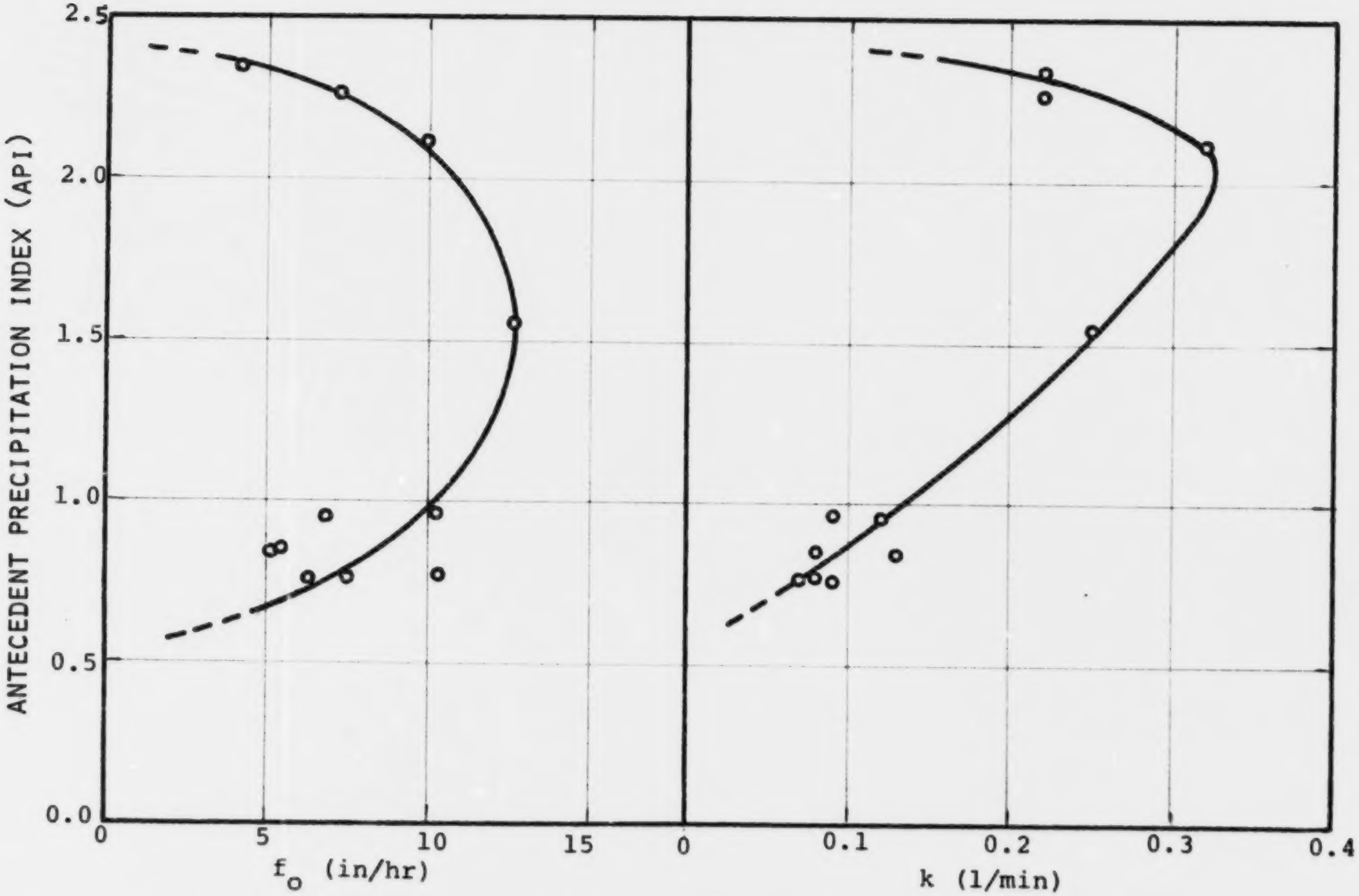

Figure 4-3. RELATIONSHIP BETWEEN API AND f_o, k OF HORTON'S EQUATION, FOR LONGVIEW SITE, BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

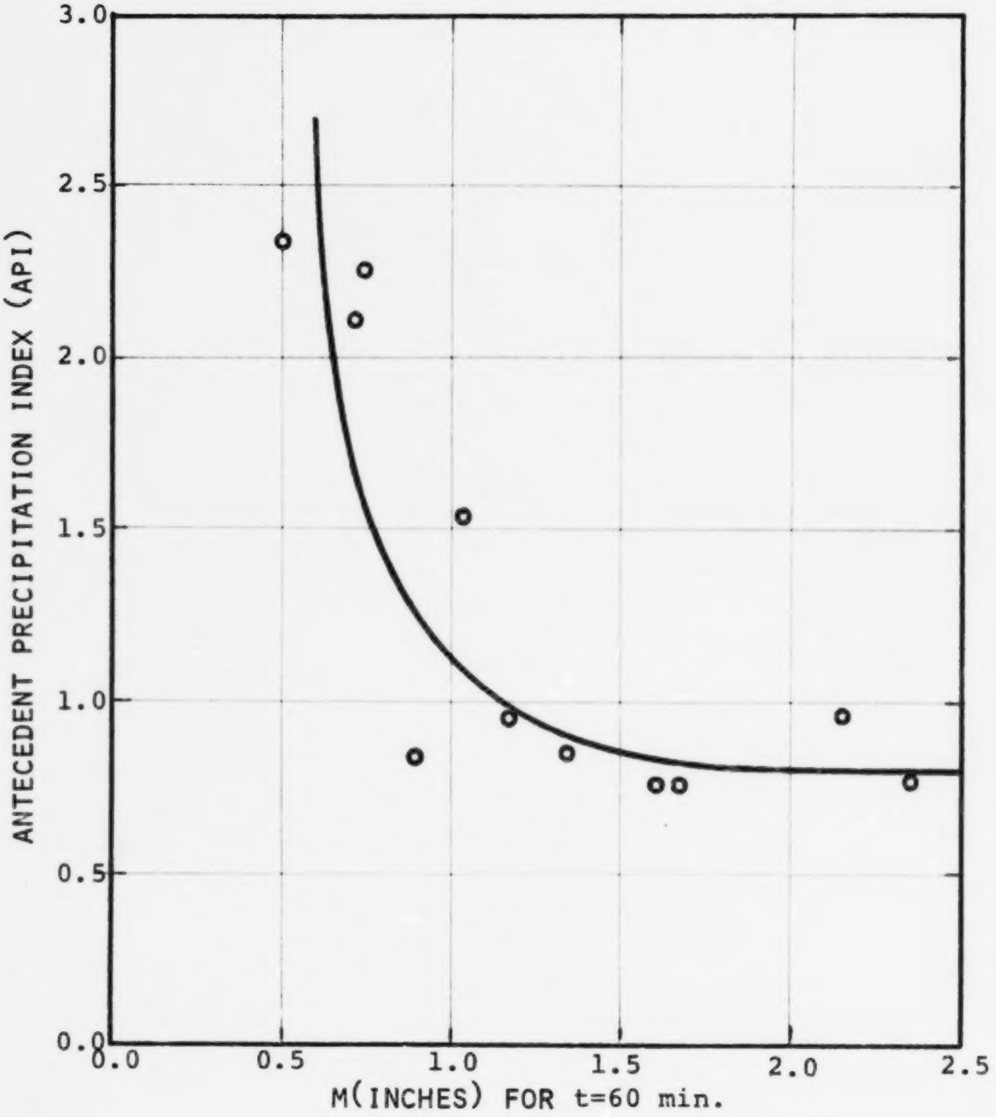

Figure 4-4. RELATIONSHIP BETWEEN API AND ACCUMULATED MASS INFILTRATED UP TO A TIME OF 60 MIN. LONGVIEW SITE, BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

moisture content of the ground is less.

If the two curves of Figure 4-3 are constructed for a given soil, for any API value an f_o and a k value can be selected. Since the constant infiltration capacity f_c varies very little, it can be taken as the average of the values recorded during the tests. Therefore for any API, Horton's equation is completely defined and can be used more successfully for the simulation of the infiltration process.

CONCLUSIONS

One of the major reasons for knowing infiltration capacity is to make accurate runoff predictions before extensive rainfall and runoff data have been collected. One way of determining infiltration capacity is by analyzing the runoff hydrographs resulting from rainstorms on a drainage basin. This method usually gives good results but has the drawback that runoff records should be available.

A simple, inexpensive and expedient method has been presented for determining the initial infiltration capacity, the decay rate of infiltration, and the constant infiltration capacity to be used in Horton's equation for a given soil and for any antecedent weather conditions.

The method consists of performing a series of infiltration tests using flooding-type tube infiltrometers and fitting Horton's equation to the available data thus obtaining values of f_o, f_c and k for each test. Then daily rainfall records should be used to determine the antecedent precipitation index (API) values starting at least two months prior to the tests. Finally, API values should be plotted against f_o and k. An average value for f_c may be used.

Urban runoff mathematical models discretize an urban watershed into a number of impervious and pervious catchments possibly with different soil characteristics. To model the infiltration process successfully, infiltration tests should be performed in several catchments of different soil type providing realistic values of Horton's constants. To minimize the influence of erroneous data a substantial number of infiltration tests per infiltrometer is recommended.

The proposed method provides the proper values of f_o k and f_c to be used in Horton's equation for the soil on which tests were performed and for any specified antecedent condition.

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SECTION 5

UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL

SYNOPSIS

Large expenditures are continuously made for the design and construction of stormwater systems for urban runoff. Yet rather crude analytical methods are often used for such design. A survey of most municipal engineering departments would likely show that the rational formula is most commonly used.

Other approaches for calculating urban runoff may be found in the literature, but in general they seem to have limitations or complications which drive the design engineer back to the use of the rational formula. It is apparent that a reliable and uncomplicated method of calculating runoff is needed for practical purposes. The method must allow calculations based on parameters which can be readily determined from limited field investigations and generally available hydrologic data. The University of Cincinnati Urban Runoff Model (abbreviated UCUR Model) described herein is believed to meet these requirements.

The UCUR Model is intended for urban watersheds of at least several hundred acres or more in area, however, it may be applied to any size through appropriate scaling. The UCUR Model Computer Program along with instructions for its use are given and examples for practical application are presented in the appendices.

DESCRIPTION OF THE UCUR MODEL

The UCUR Model consists of five sub-models which simulate individually the processes involved in runoff. They include:

1. Infiltration Sub-Model
2. Surface Retention Sub-Model
3. Overland Flow Sub-Model
4. Gutter Flow Sub-Model
5. Sewer Routing Sub-Model

These sub-models are combined in sequence through the use of a computer to calculate the runoff hydrograph at the system outlet or at selected points within the system. The format of the calculation procedure follows that shown in the flow chart of Figure 5-1.

Basically, the procedure is to simulate specific hydrologic conditions using a deterministic method with a parametric system synthesis approach [1]. Input parameters include the rainfall hyetograph and physical data which describe the watershed characteristics and the drainage system.

Initially, the total drainage basin is divided into a number of catchments having an area of approximately 10 to 20 acres; larger catchments may be used, but with some relative loss in accuracy. Each catchment is then further divided into a number of subareas or subcatchments which should in general have uniform slope and be identified with regard to cover. This process can best be accomplished using topographic maps and/or aerial photographs. In lieu of either of these sources of information, it would be necessary to closely inspect the watershed and develop catchments and subcatchments accordingly. The procedure of subdividing the area is quite flexible and is only limited by manpower requirements and computer storage availability. A large number of subdivisions will require more computer storage than a smaller one and of course more computer time will be needed in executing the program. Theoretically, the subdivision procedure could be carried to infinitesimal detail (i.e. the roof of a house could be represented by two subcatchments each sloping in an opposite direction). However, some judgment must be used so as to develop a pattern which is consistent with the accuracy of other input parameters.

The developed computer program accepts as input any design storm or actual storm hyetograph. However, in order to select a design storm pattern (Section 3), consideration should be given to the fact that high-intensity, short-duration rainfall is normally the main, if not sole, type of precipitation contributing to critical runoff rates. This type of rainfall is usually associated with thunderstorms. Intensity-duration-frequency data of such rainfall in the United States were first analyzed by Yarnell [27] , but are now being modified and improved with additional data by the U.S. Weather Bureau .

The UCUR Model can accept a different rainfall pattern over each catchment if a sufficient number of raingages are located on the watershed so as to define the intensity variations. However, the rainfall pattern has to be considered uniform over each catchment. This is a reasonable

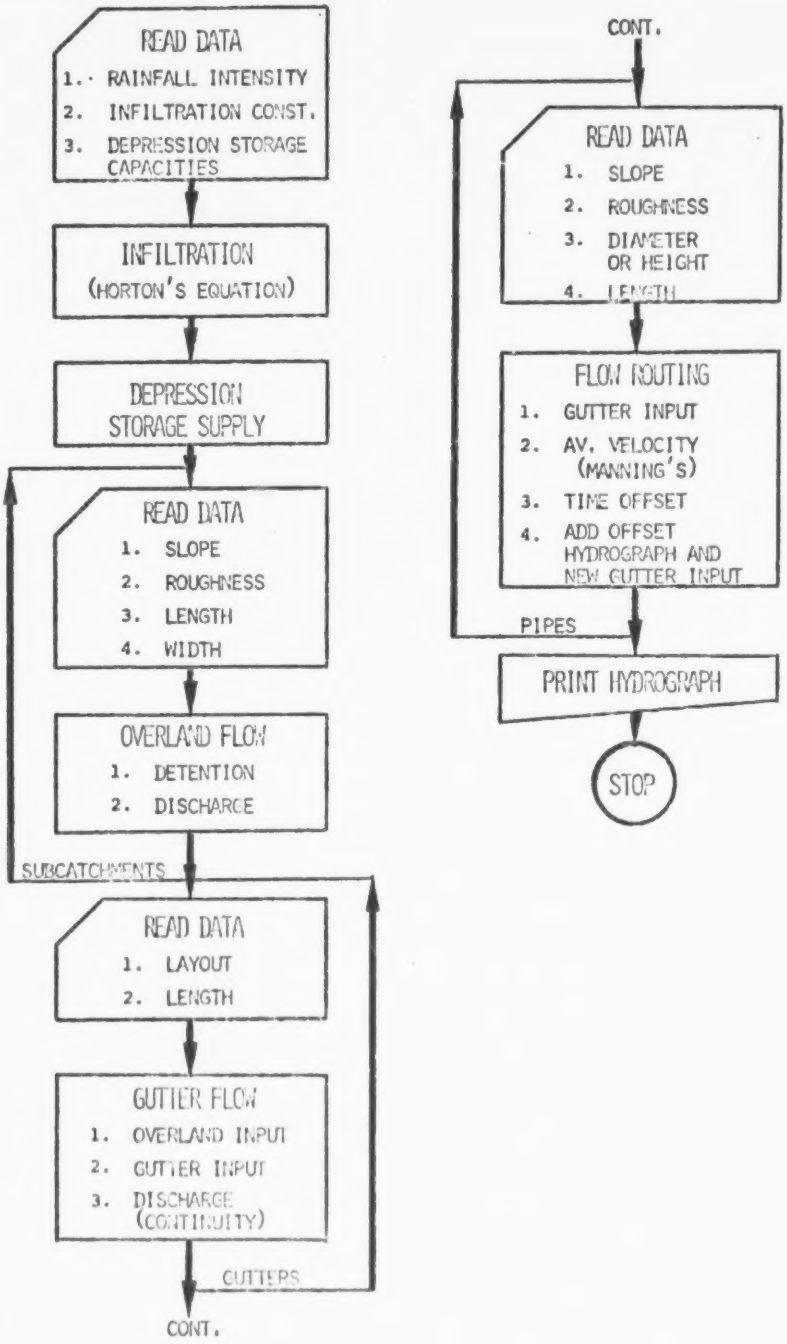

READ DATA
1. RAINFALL INTENSITY
2. INFILTRATION CONST.
3. DEPRESSION STORAGE CAPACITIES
INFILTRATION
(HORTON'S EQUATION)
DEPRESSION STORAGE SUPPLY
READ DATA
1. SLOPE
2. ROUGHNESS
3. LENGTH
4. WIDTH
OVERLAND FLOW
1. DETENTION
2. DISCHARGE
SUBCATCHMENTS
READ DATA
1. LAYOUT
2. LENGTH
GUTTER FLOW
1. OVERLAND INPUT
2. GUTTER INPUT
3. DISCHARGE (CONTINUITY)
GUTTERS
CONT.
CONT.
READ DATA
1. SLOPE
2. ROUGHNESS
3. DIAMETER OR HEIGHT
4. LENGTH
FLOW ROUTING
1. GUTTER INPUT
2. AV. VELOCITY (MANNING'S)
3. TIME OFFSET
4. ADD OFFSET HYDROGRAPH AND NEW GUTTER INPUT
PIPES
PRINT HYDROGRAPH
STOP

Figure 5-1. UCUR MODEL COMPUTER FLOW CHART

assumption considering the relatively small area of the catchments.

The five sub-models which make up the UCUR Model simulate individually the processes involved, i.e., infiltration, surface retention, overland flow, gutter flow, and routing through the sewer system. Brief descriptions are given below on the analytical approaches for each of these sub-models:

1. The Infiltration Sub-Model employs a well known equation by Horton [13] to determine the proper shape of the infiltration capacity curve. A method has been developed for shifting the infiltration capacity curve with respect to time so that Horton's equation can be used when the initial rainfall intensity is less than the initial infiltration capacity [24]. If actual data are not available for the urban watershed under study, standard infiltration capacity curves by Jens [17] can be substituted, or Horton's constants can be evaluated based on a few flooding-type tube infiltrometer tests and the corresponding antecedent precipitation index (Section 4).

2. For the Surface Retention Sub-Model, an exponential relationship has been developed which relates depression storage supply with time assuming that the rate with which depression storage is filled is proportional to the unfilled volume. The basis for the equation has been derived from an exponential relationship proposed by Linsley, Kohler, and Paulhus [20].

3. The development of the Overload Flow Sub-Model is based on the conservation of mass and the continuity equation. An equation by Crawford [5] has also been used which relates the overland flow depth with the surface detention. Ground slope, cover, and the length of overland flow are significant variables in this process.

4. For the Gutter Flow Sub-Model, the continuity equation has again been employed assuming steady flow conditions. This assumption is believed to be justified since relatively slow under most runoff conditions [24].

5. In the Sewer Routing Sub-Model, routing through the laterals and main sewer is accomplished by shifting the inflow hydrograph from one inlet to the next downstream, thence from the sewer laterals to the sewer main. Each preceding hydrograph is added to the next junction downstream until the full system is encompassed at the final outlet. Flow through the piping system is based on Manning's equation. Actually, the method results in peak flows which are slightly higher than

those calculated by using more exact methods. However, the simplicity of the procedure is an advantage over more rigorous methods such as backwater curves computation upstream from the outlet.

The specific development for each of the sub-models and the demonstration of the practical utility of the overall procedure are provided in the following.

THE INFILTRATION SUB-MODEL

Infiltration is the process during which water enters the surface strata of the soil and moves downward toward the water table. The maximum rate at which a soil in any given condition is capable of absorbing water is called its infiltration capacity. It is considered that infiltration capacity in pervious subcatchments follows Horton's exponential equation [13]:

$$f = f_c + (f_o - f_c)\, e^{-K\, t_f} \tag{1}$$

where

f = infiltration capacity in inches per hour.

t_f = time in minutes elapsed from the beginning of the infiltration capacity curve.

f_c = the constant rate which f is approaching asymptotically as time continues.

f_o = initial infiltration capacity when t_f=0.

K = decay rate of infiltration which is a positive constant, with (1/minute) units.

The main advantage of equation (1) is that f_o, f_c, and K may be related to physical characteristics of the soil types for which curves are available, and this may therefore eventually make possible predictions of the infiltration capacity of soil types for which no experimental data have been obtained [14]. If it is desired to use a constant infiltration capacity throughout the basin under consideration, equation (1) for $f_c = f_o$ becomes $f = f_c$, independent of time. If there are no data available concerning infiltration capacity for the catchment under study, K, f_c, and f_o can be evaluated from the infiltration capacity curves by Jens [17] which have been made for turfed areas of airfields and have been used widely for city lawns. Evaluations of infiltration capacity for different soils and cover classifications can also be found in the Soil Conservation Service Handbook [22] and in the ASCE Hydrology Handbook [14].

Antecedent precipitation plays an important role in the shape of the infiltration capacity curves. Under wet antecedent conditions a soil will present a decreased constant infiltration capacity and the infiltration capacity curve will drop more rapidly. This gave rise to the idea of relating f_o and K with the antecedent precipitation index (Section 4). However, in the case of impervious areas, the only importance of antecedent precipitation is in wetting the surface and filling depressions. House roofs, streets, and mechanically compacted soil are considered as having negligible infiltration capacity.

The area under the infiltration capacity curve represents the accumulated mass infiltrated, which is:

$$F = \int_0^{t_f} \frac{f}{60}\, dt$$

$$F = \frac{f_c}{60} t_f + \frac{(f_o - f_c)}{60\ K} \left[1 - e^{-Kt_f} \right] \qquad (2)$$

Where F is the total depth in inches infiltrated up to the time t_f.

If the rain starts and continues with an intensity i greater than f, then time t_f in equations (1) and (2) is also the time t in minutes from the beginning of the rainfall. Therefore, the infiltration capacity curve and the rainfall hyetograph start together. However, if at the beginning of the rainfall, the intensity is less than the infiltration capacity, then the actual infiltration rate is less than the infiltration capacity and the infiltration capacity curve will be offset by an amount of time t_o. Then t_f in equations (1) and (2) is the time from the beginning of the infiltration capacity curve, and the corresponding time t from the beginning of the rain now becomes $t = t_o + t_f$. In this case, when the rain starts, the actual infiltration curve follows the rainfall intensity curve until it intercepts the offset infiltration capacity curve at a point A, following which it proceeds along the latter, as shown in Figure 5-2.

Assuming that overland flow and the depression storage supply commence when the antecedent volume of rainfall equals the antecedent volume of infiltration, the offset time t_o can be defined as the time necessary for (area BAGH) = (area EAGF). Actually two conditions must be satisfied at the point of intersection A:

1. [rainfall intensity at A] = [infiltration rate at A]
2. [accumulated volume precipitated at A] = [accumulated volume infiltrated at A].

The solution of the system generated from the above conditions by digital computer [24] or graphically [26] yields the time t_o. Computation of the actual infiltration curve can then proceed and the stored value of t_o is used to start the calculation of the surface retention.

THE SURFACE RETENTION SUB-MODEL

Surface retention includes interception, depression storage, and evaporation during a storm. Interception by vegetation may range from 0.01 to 0.5 inches in forest areas, but is not usually significant in urban drainage. Evaporation is of little significance for the short rainfall duration encountered in urban storm drainage design. Therefore, for simplicity of the model, interception and evaporation are assumed negligible for an urban area.

Some investigators have assumed that all depressions must be filled before overland flow begins. The real situation is that almost immediately after the beginning of the rainfall excess, the smaller depressions become filled and overland flow begins. Most of this water in turn fills larger depressions, but some of it follows an unobstructed path to the collecting gutters.

Linsley, Kohler, and Paulhus [20] recommended an exponential relationship

$$V_s = S_d \left[1 - e^{-N(P-F)} \right] \tag{3}$$

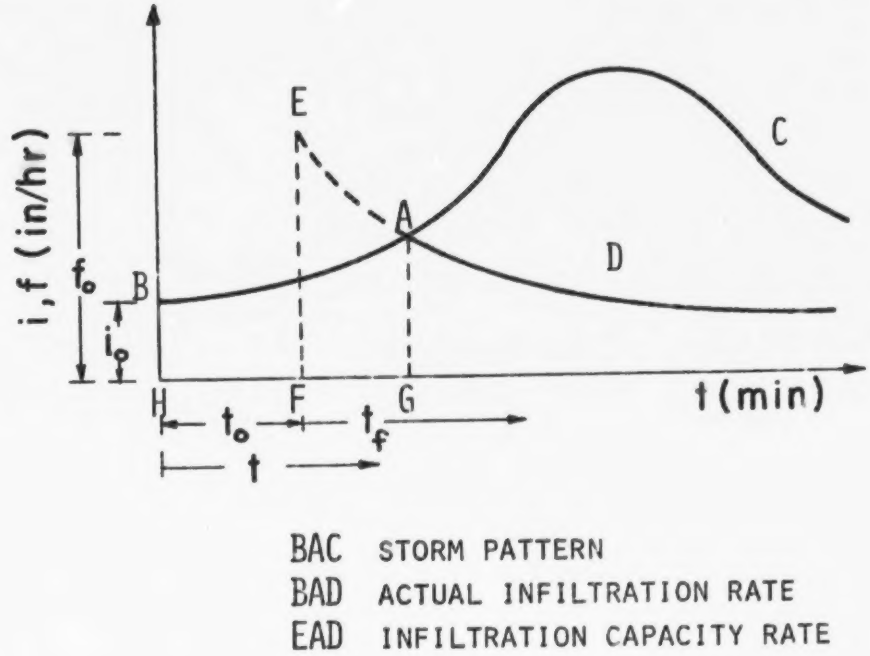


Figure 5-2

STORM PATTERN AND INFILTRATION CAPACITY IN CASE $i_o < f_o$

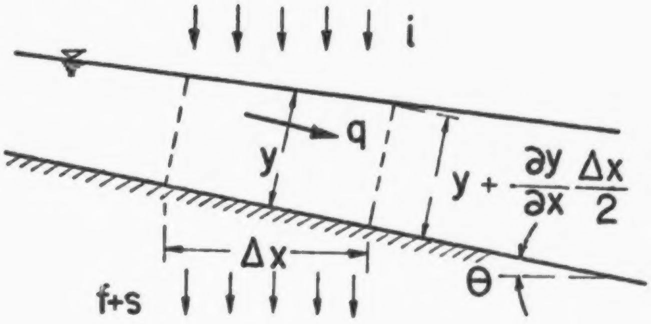

i
q
y
$y + \frac{\partial y}{\partial x}\frac{\Delta x}{2}$
Δx
θ
f+s

Figure 5-3

OVERLAND FLOW

where V_s = volume of water in depression storage at any time throughout a storm, in inches.

S_d = total depression storage capacity of the basin, in inches.

P = accumulated volume of water precipitated in inches.

F = accumulated volume of water infiltrated in inches.

N = storage constant.

Assuming that the initial increment of rainfall excess is completely retained by depressions:

$$\frac{dV_s}{d(P-F)} = 1 \quad \text{when } (P-F) \text{ is near zero.}$$

Then by differentiation:

$$N = \frac{1}{S_d} \tag{4}$$

If the depression storage supply is s (in/hr):

$$\int_0^t s\, dt = V_s$$

where t is time in hours from the beginning of the rainfall excess.

Then $$s = \frac{d}{dt}\left[S_d - S_d\, e^{-\frac{P-F}{S_d}} \right]$$

or $$s = \frac{d}{dt}\left[(P-F)\, e^{-\frac{P-F}{S_d}} \right]$$

but $$\frac{d}{dt}(P-F) = (i - f) \qquad \text{(in/hr)}$$

therefore $$s = (i - f)\, e^{-\frac{P-F}{S_d}} \tag{5}$$

On impervious areas where infiltration is assumed equal to zero and also accumulated volume of water infiltrated does not exist, equations (3) and (5) become:

$$V_s = S_d \left[1 - e^{-\frac{P}{S_d}} \right]$$

$$s = i\, e^{-\frac{P}{S_d}}$$

Horton [11] stated that the depression storage commonly ranges from 0.125 to 0.75 inches for flat areas and from 0.5 to 1.5 inches for cultivated fields and for natural grass lands or forests. On moderate or gentle slopes he estimated that pervious surface depressions can commonly hold the equivalent of 0.25 to 0.5 inches of water and even more on natural meadow and forest land. Hicks [9] has used depression storage losses of 0.20 inches for sand, 0.15 inches for loam, and 0.10 inches for clay, considering high rates of rainfall and runoff. Recent gagings show depression storage to be about 0.3 inches in forest litter, 0.2 inches in good pasture, and 0.05 to 0.10 inches in smooth cultivated land [6].

If there are no data available, the overall depression storage capacity can be assumed equal to 0.25 inches on pervious urban areas which usually present individual depressions with depths from 0 to 0.5 inches. The overall depression storage on impervious urban areas can be assumed equal to 0.0625 inches with the depth of depressions ranging from 0 to 0.125 inches. These assumptions, when there is a lack of data for the basin under consideration, seem reasonable from observations made during rainfalls [18], [21], [26].

The rainfall excess which produces the overland flow is obtained by subtracting the calculated values of infiltration and depression storage supply from the rainfall intensity at any time.

THE OVERLAND FLOW SUB-MODEL

Because of the complex nature of the overland flow, models are generally simplified to two dimensions. This was the approach used by Keulegan [19] who, in the year 1944, first worked on the spatially varied unsteady flow problem. Horton [12] and Izzard [16] studied unsteady flow across sloping plane surfaces. However, Horton's equation for overland flow has limited experimental support, and Izzard's dimensionless hydrograph is limited to situations where iL is less than 500, where i is the rainfall intensity in inches per hour and L is the length of the overland flow in feet.

By subtracting infiltration and surface retention from the rainfall intensity, the overland flow supply is obtained. As this supply increases, a sheet of water builds up over the surface and the water starts to flow toward the collecting gutters. This movement of water is overland flow and the volume of water on the surface is the surface detention.

Considering flow of unit width on a surface with small bottom slope ($\sin \theta = \theta$) and uniform velocity distribution (Figure 5-3), the continuity equation can be written:

$$\frac{\partial q}{\partial x} \Delta x + \frac{\partial A}{\partial t} \Delta x = i \; \Delta x - f \; \Delta x - s \; \Delta x$$

or

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = r \tag{6}$$

where y = depth of flow in feet

q = discharge in cfs/ft of width

r = (i-f-s) the inflow or supply rate in cfs/sq.ft.

Field investigations have shown that overland flow in thin sheets may be entirely turbulent, or partly turbulent and partly laminar. For smooth surfaces, it appears that laminar flow exists with the possibility of changing from laminar to turbulent, and vice versa, within a short distance. For subdivided flow through grass, most of the experimental data indicate that the flow is mainly turbulent in which case the profile of overland flow can be assumed approximately parabolic [10]. Experimental measurements of surface detention show also that high intensity rainfalls often yield Reynold's numbers that indicate turbulent flows. Because the division between the laminar and turbulent ranges is difficult to establish and because the inclusion of viscosity is considered difficult, overland flow is assumed to be turbulent and to be governed by the acceleration of gravity.

For turbulent flow, Manning's equation can be used, considering the hydraulic radius R equal to the depth y:

$$q = 1.49 \frac{\sqrt{\sin\theta}}{n} y^{5/3} \tag{7}$$

Equation (7) is of the general form:

$$q = C\, y^{m} \tag{8}$$

In practice, equation (8) is a good approximation for the discharge from surface flow. This conclusion is supported by Keulegan [19], Chow [2], and Eagleson [7].

Equations (7) and (8) constitute a kinematic wave problem which has been solved by Eagleson [7] for $s = 0$ and i and f constants in time and space, using the method of characteristics. The only rigorous general methods for simulating unsteady overland flow are finite difference techniques for the numerical solution of the governing partial differential equations. However, the accuracy to be gained by using such laborious methods for overland flow is subject to question because of the limited accuracy of the basic field data. Therefore, an analytical method based on the time necessary to reach equilibrium and on an empirical relationship between outflow depth and detention storage [5] has been chosen in connection with a routing equation.

Between time zero and the time to equilibrium, the total inflow will be equal to the total runoff (outflow) plus the surface detention (storage). This relation yields [5], [24]:

$$D_e = \frac{0.009792\, r^{0.6}\; n^{0.6}\; L^{0.6}}{S^{0.3}} \tag{9}$$

where: D_e = detention storage at equilibrium in inches per unit area

r = overland flow supply rate in inches per hour

S = slope of the plane in ft/ft

L = length of overland flow in feet

n = Manning's roughness coefficient

The most satisfactory empirical relationship found between outflow depth and detention storage for reproducing experimental hydrographs [5] is:

$$y = D \left[1.0 + 0.6 \left(\frac{D}{D_e}\right)^{3} \right] \tag{10}$$

where y = outflow depth in inches

D = current detention storage in inches per unit area

Then Manning's equation becomes:

$$q = \frac{1020.7}{n\ L}\ S^{1/2}\ D^{5/3}\left[1.0 + 0.6\ \left(\frac{D}{D_e}\right)^3\right]^{5/3} \qquad (11)$$

with units q in inches per hour per unit area, L in feet and D, D_e in inches per unit area

To determine the overland flow runoff hydrograph, a storage routing procedure is used. The storage equation is:

$$r - q = 60\ \frac{dD}{dt}$$

where r is the inflow, q is the outflow, dD is the change in detention storage, and dt is the time increment in minutes.

This equation can be written in an incremental form [8]

$$\frac{r_1 + r_2}{2} - \frac{q_1}{2} + 60\ \frac{D_1}{\Delta t} = \frac{q_2}{2} + 60\ \frac{D_2}{\Delta t} \qquad (12)$$

where the subscripts 1 and 2 indicate values at the beginning and the end of the routing increment Δt.

Combining equation (12) with equation (11):

$$\frac{r_1+r_2}{2} - \frac{q_1}{2}+60\frac{D_1}{\Delta t}=\frac{510.35}{nL}S^{1/2}D_2^{5/3}\left\{1.0+0.6\left[\frac{D_2}{D_{e_2}}\right]^3\right\}^{5/3} +60\frac{D_2}{\Delta t} \quad (13)$$

where D_e can be found for each corresponding r from equation (9).

All the known values are now at the left-hand side of equation (13). Starting at the first time increment with $q_1 = 0$, $D_1 = 0$ the value of D_2 is found from equation (13) and then q_2 is found from equation (11). Then the calculated values of q_2 and D_2 become q_1 and D_1 for the subsequent period and the above process is repeated giving the hydrograph of the overland flow in inches per hour which can be directly compared to the rainfall rates. Equations (11) and (13) are non-linear and the Newton-Raphson's iterative method is used for their solution.

However, in order to calculate the discharge entering the gutters due to the overland flow supply, it is more convenient to have available the overland flow in cubic feet per second per foot of width. This can be done by multiplying the values of q found from the routing procedure by a factor, i.e.

$$q\left(\frac{\text{cfs}}{\text{ft width}}\right) = \frac{L\ (\text{ft})}{43200}\ q\left(\frac{\text{in/hr}}{\text{unit area}}\right)$$

Where L is the length of the overland flow.

This method has the advantage of using Manning's roughness coefficient for which there are much experimental data relating to different types of ground cover [2], [5]. The overland flow model has been found to agree satisfactorily with the well known Izzard method [15], [16]. Some differences occurred probably due to the fact that Manning's n= 0.35 used in the comparisons, may not well correspond to the c= 0.06 used by Izzard [24].

Testing has shown that the overland flow model adequately follows the continuity principle. The sensitivity of the model is demonstrated in Figure 5 -4 when subjected to variations of the Manning's roughness coefficient and of the slope and the length of the overland flow plane. For increasing Manning's roughness coefficients, the peaks of the overland flow hydrographs tend to be higher and to occur at earlier times. For increasing lengths of overland flow, the peaks tend to be lower and to occur at a later time, resulting in a flatter overland flow hydrograph.

The method of computing overland flow as presented above assumes steady flow within each time interval (quasi-steady state). Therefore the accuracy of the solution would decrease for larger time intervals. To demonstrate the reliability of the UCUR Overland Flow Sub-Model a case study is presented in the following. A rectangular concrete area having a Manning's roughness coefficient of 0.012 is subjected to a rainfall excess of 1.70 in/hr for the first 6 min., 3.77 in/hr for the next 12 min., and 1.75 in/hr for the last 14 min. The length and slope of the area parallel to the direction of overland flow are 252 ft and 0.005 ft/ft. respectively. The kinematic

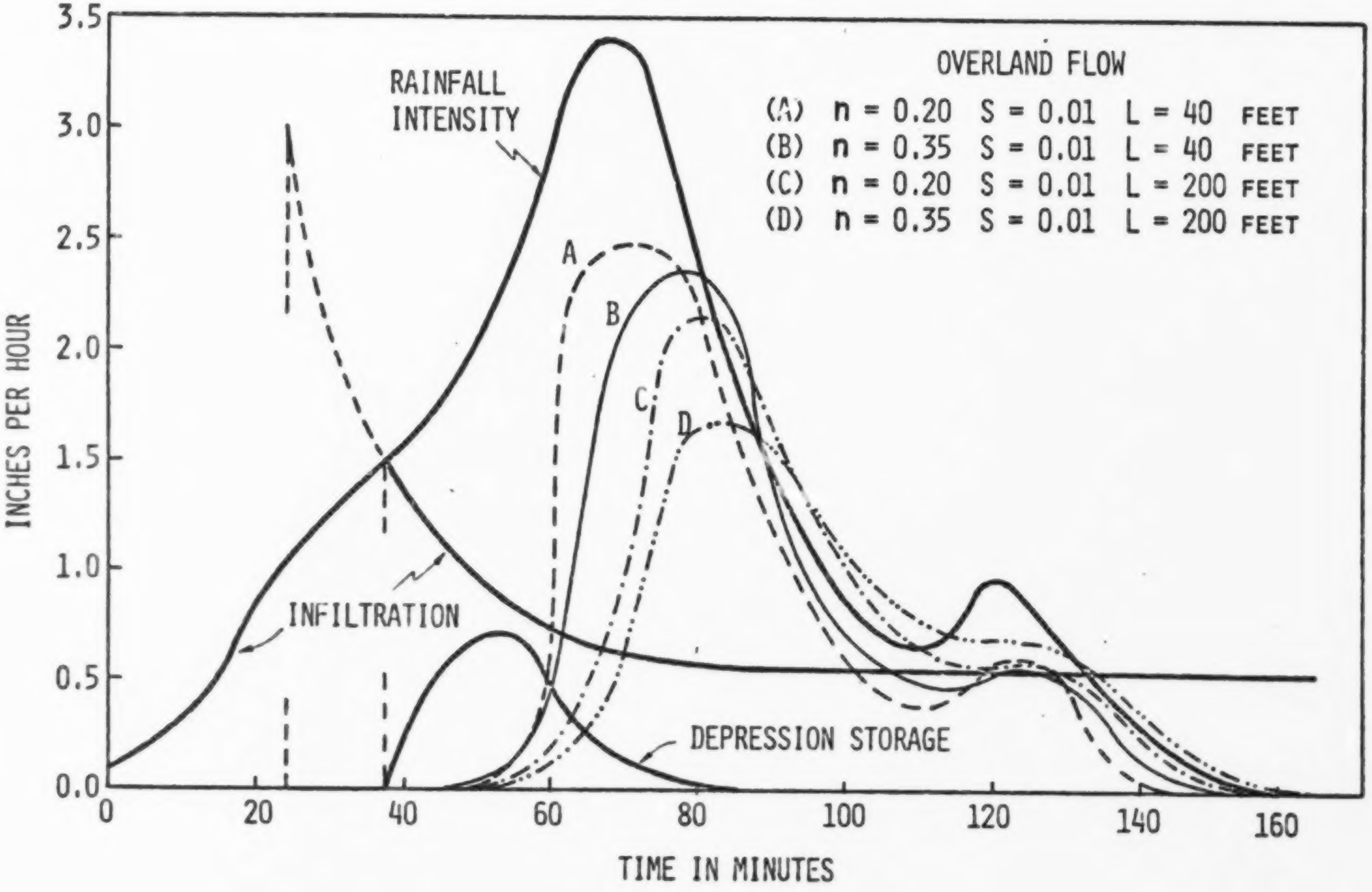


Figure 5-4.

SENSITIVITY OF THE OVERLAND FLOW SUB-MODEL wave theory, the EPA Storm Water Management Model and the UCUR Overland flow submodel were used to calculate the hydrograph of overland flow at the downstream end of the basin per unit width. The results are plotted in Figure 5-5 together with the recorded hydrograph. The best comparison was obtained by using the kinematic wave theory, as expected. The UCUR submodel produced a quite satisfactory simulation with a considerable saving in the computer time involved. This case study was performed by James F. MacLaren Ltd., Toronto, Canada.

The overland flow model as developed offers simplicity and calculating speed compared with other more exact methods, yet it maintains a reasonable approximation of physical behavior.

THE GUTTER FLOW SUB-MODEL

Gutters gather overland flow continuously along their length to feed the catchment inlets and catch basins. By introducing equal increments of flow at regular intervals along the length of a gutter it has been shown experimentally that a difference exists between the water-surface slope and the construction grade. The upstream water slope is much flatter.

The type of flow in roadside gutters is spatially varied unsteady flow with increasing discharge. In this type of flow, the energy loss is of relatively high magnitude and uncertainty, and is due to the turbulent mixing of the added water and the water flowing in the channel[8]. The momentum equation would be found to be more correct than the energy equation. However, because only the discharge entering the sewer inlets at the downstream end of the gutters is required, the momentum equation has not been used and the discharged Q at the end of the gutters is obtained by using only the continuity equation for unsteady flow with lateral inflow q_L:

$$\frac{\partial Q}{\partial x} + \frac{\partial y}{\partial t} T = q_L \qquad (14)$$

The lateral inflow q_L, available from the overland flow, is considered to be constant along the length of the gutter for a time interval. Therefore, q_L is a function of t but not of x. In the above equation, T is the width of the water surface.

For an urban area, the lengths of the gutters are relatively small. Also the overland flow supply is small. Therefore, in a time increment, the change of the depth in the

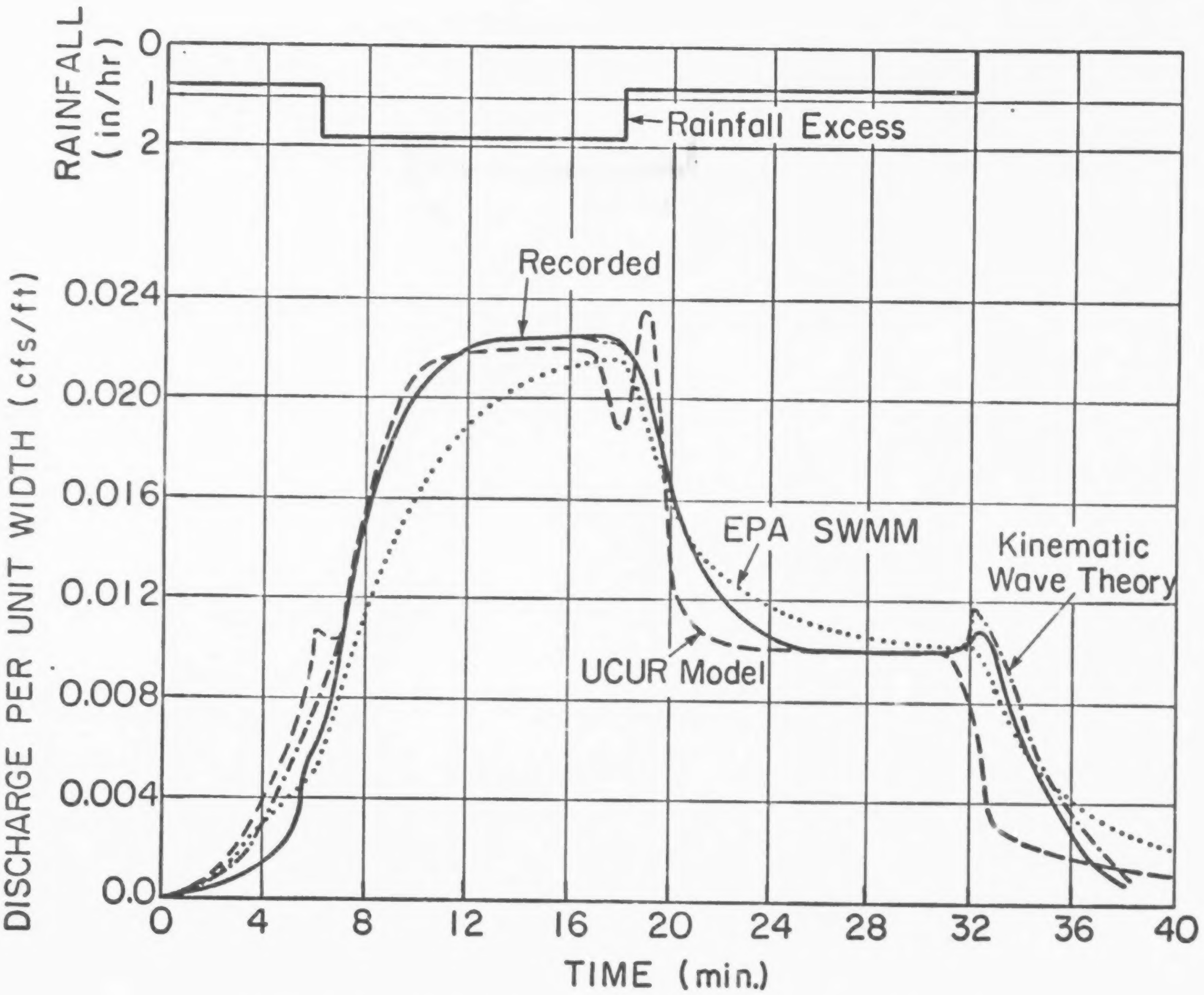


FIGURE 5-5. RECORDED AND CALCULATED OVERLAND FLOW HYDROGRAPHS

gutter was found to be very small in comparison with the other terms of equation (14) [25]. Omitting the term $\frac{\partial y}{\partial t}$ T equation (14) becomes:

$$\frac{\partial Q}{\partial x} = q_L$$

and integrating: $Q = q_L L + Q_o$ (15)

where Q_o is the discharge entering the gutter from an upstream gutter, otherwise zero (Figure 5-6). It is evident that in cases where storage effects become predominant (flat gutter grades or high rates of overland flow) the above approach would lose in accuracy.

Once the hydrograph of the discharge entering into an inlet (inflow hydrograph) is obtained from equation (15) it will be routed through the sewer until the next inlet downstream, where a new inflow hydrograph will be added to the routed one and the summation will be routed downstream, and so on.

THE SEWER ROUTING SUB-MODEL

In Chicago, Tholin and Keifer [26] used a time-offset method for conduit routing because of its simplicity compared with other routing methods and because more refined procedures (storage routing) had not led to significantly different results. Storage routing can be applied if satisfactory discharge-storage relationships are available. This makes necessary the computation of instantaneous backwater curves. Since only the rate of change in storage is necessary to solve the storage equation, it is considered expedient to assume a uniform flow condition for each discharge rate [2] and to compute the conduit volume occupied by the flow. Other methods used in Europe [23], such as the semigraphical method by Hauff and the Italian storage method, are relatively easy to use, but are based on broader assumptions.

Generally there are many points of entry (inlets and/or catch basins) into a lateral sewer. The junctions with the lateral sewers are considered as points of entry for a main sewer. For each point of entry, a hydrograph of the inflow is available from the gutter flow sub-model. The computer automatically transforms these smooth curve inflow hydrographs into histograms having constant discharge rates for each time increment of 1 minute. Assuming a uniform flow condition for each discharge rate, Manning's equation can be

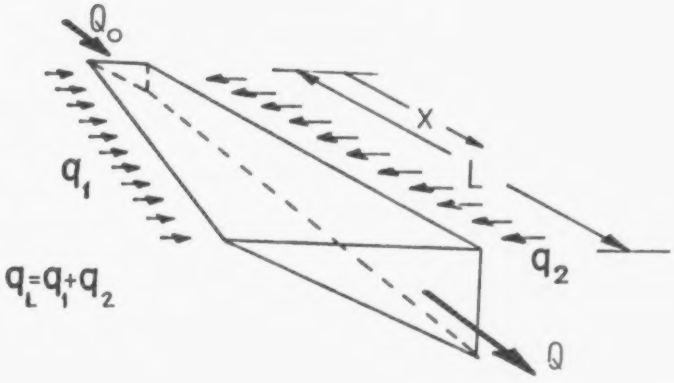

Q_o
x
L
q_1
q_2
$q_L = q_1 + q_2$
Q

Figure 5-6.
GUTTER FLOW

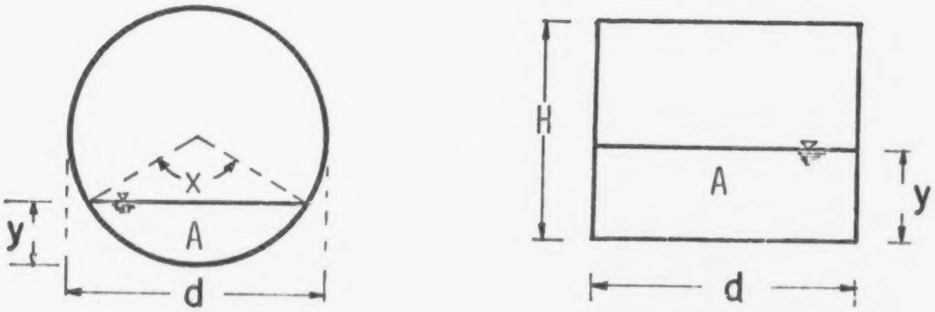


Figure 5-7.
COMMON SEWER CROSS-SECTION

used to find the depth and the velocity of flow into the sewer for each discharge. Therefore, the velocity with which each water volume element travels downstream can be found. The average velocity with which the total inflow volume (represented by the area under the hydrograph) moves downstream is considered to be the weighted average of all the partial velocities with respect to the corresponding volumes (i.e. multiply each volume element by its velocity, sum the products and divide the sum by the total volume of the water which moves downstream). Since the average velocity of flow is found and the length of the conduit between inlets is known the time required for the inflow water volume to travel this distance may be found.

Each inflow hydrograph is shifted in time without changing in shape. This has been found satisfactory and convenient, producing only a slightly higher peak rate of flow occurring at a somewhat later time than other more exact methods. Therefore, for a lateral sewer of uniform slope and cross-section, the computer shifts the first inflow hydrograph at the time required to reach the second inlet. Then, it adds the new inflow hydrograph and the shifted one and shifts the summation hydrograph until the next inlet, and so on until it reaches a critical point. Changes in slope or changes in cross-section are considered as critical points. After a critical point the procedure is repeated from the beginning considering as first inflow hydrograph the one found at the critical point. This continues to the junction of the respective lateral sewer with a main sewer. Routing through the lateral sewers is done first in order to find the hydrographs entering the main sewer at the junctions. These hydrographs are used as inflow hydrographs for the main sewer and the same method is followed to find the hydrographs at selected points in the main sewer.

Surcharging of the sewer system is not accounted for. The computer compares continuously the maximum capacity of the conduits and the routed discharges. If the capacity of a conduit is not sufficient, the computer indicates that in the output. Therefore, in the case of full flow, the cross-section of the insufficient conduit or its slope or both should be changed.

It is assumed that the Manning's roughness coefficient remains constant as the depth changes although actually the roughness coefficient decreases as the depth of flow increases. For practical purposes, it can also be assumed that the maximum discharge of a circular conduit occurs at its full depth since maximum discharge occurs only slightly below the crown [2].

The pipe flow is treated as a quasi-steady state process (flow assumed steady within each Δt). In order to find the velocity with which the volume ($Q\,\Delta t$) is traveling downstream, Manning's turbulent flow equation is used:

$$Q = \frac{1.486}{n} A\, R^{2/3}\, S^{1/2} \tag{16}$$

where: Q = discharge in cfs entering into the sewer during a time increment Δt, known from the inflow hydrograph.

n = roughness coefficient of the conduit, usually 0.013 [3] and [4].

A = cross-sectional area of flow in square feet.

R = hydraulic radius in feet.

S = slope of the invert of the conduit in ft/ft.

The present method is used for sewers of circular and rectangular cross-section (Figure 5-7). Other shapes can be added to the computer program or can be approximately transformed into these two basic cross-sections. For partially full flow conditions in circular pipes, equation (16) becomes:

$$\frac{(x - \sin x)^{5/3}}{x^{2/3}} = \frac{13.566\, Q\, n}{d^{8/3}\quad S^{1/2}} \tag{17}$$

For partially full flow in rectangular pipes, equation (16) becomes

$$\frac{y^{5/3}}{(d + 2y)^{2/3}} = \frac{Q\, n}{1.486\, S^{1/2}\, d^{5/3}} \tag{18}$$

The right hand part of equations (17) and (18) is known, therefore these two equations can be solved by using Newton-Raphson's iterative method to find the angle x in radians or the depth of flow y in feet. Then the velocity of flow can be found by using the equation $V = Q/A$.

If an inflow hydrograph consists of N time increments t with the corresponding discharges Q_1, Q_2, ..., Q_N, then the average velocity with which the total inflow volume travels downstream until the next inlet is:

$$V_{av} = (V_1Q_1 + V_2Q_2 + \ldots + V_NQ_N)/(Q_1 + Q_2 + \ldots + Q_N)$$

where V_1, V_2, ... are the velocities with which the volumes $Q_1\ \Delta t$, $Q_2\ \Delta t$, ... travel downstream.

The computational effort involved in calculating the weighted average velocity was considered excessive by Heeps and Mein, Monach University, Australia (reference [6] of Section 10). Hence, a simplifying modification was implemented according to their suggestion. The weighted average velocity was replaced by the velocity of flow corresponding to the centroidal discharge of the hydrograph. Heeps and Mein found that this change considerably reduced the execution time involved and never altered the timing of the hydrograph by more than one minute.

MODEL APPLICATION

The complete UCUR Model is the sequential addition of each of the sub-models just described. Since the amount of computations involved in this process is quite formidable, a computer program has been developed and is available together with detailed instructions for compiling and inserting the necessary data given in a step by step procedure. The computer program provides rapid calculation of runoff at desired points in the sewer system. To use the program, some basic knowledge of Fortran programming and the methods of inputting the data is desirable.

Initially, physical data on the watershed and drainage system must be gathered. Usually, this information can be compiled from existing engineering sources plus some additional field investigations. The extent to which these data are acquired should be consistent with the overall accuracy desired in the runoff estimates. Secondly, it is necessary to decide on the storm pattern to be considered. Any rainfall hyetograph may be used as input, and testing of the model can be achieved if the corresponding hydrograph has been recorded at the outlet of the watershed under study. Another alternative is to select a proper design storm hyetograph. This requires knowledge of the local hydrology and the allowable average frequency of overloading of the proposed sewer system. To determine the design rainfall hyetograph which may be used as input to the developed computer program, any reliable method can be implemented. In Section 3, the intensity duration curve is determined from rainfall statistics for the selected frequency of overloading and then the chronological storm pattern or hyetograph is calculated from those storms which are most likely to cause excessive runoff. A more severe condition might be the combination of two or more storms in consecutive or overlapping order. Engineering judgement must be used in making the storm selection since the UCUR Model may use any storm hyetograph as input data.

UCUR MODEL COMPUTER PROGRAM

The volume of computations required by the UCUR Model would have been too formidable without the use of a high speed computer. The UCUR Program is written in FORTRAN IV Language and was executed on the digital computer IBM 360/65 of the University of Cincinnati, which has a maximum storage capacity of 512 K.

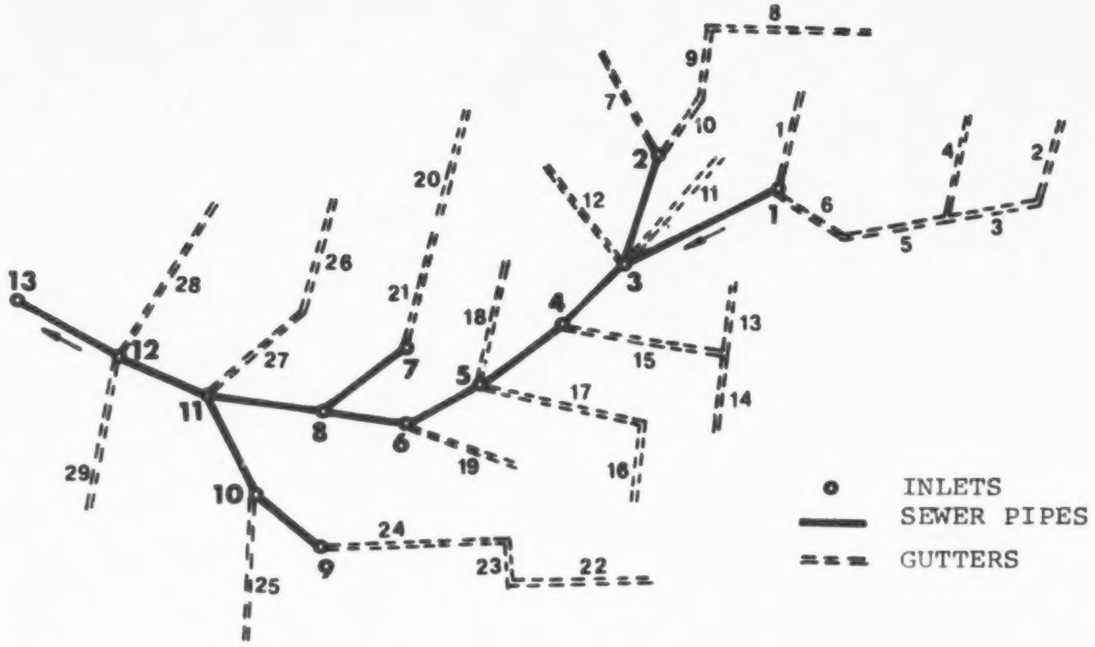
The UCUR Program consists of four parts, the main program and three subroutines, namely:

1. MAIN Program: simulation of infiltration and depression storage.
2. OVFLOW Subroutine: overland flow simulation.
3. GUTFL Subroutine: gutter flow routing.
4. PIROU Subroutine: routing through lateral and main sewers and bookkeeping for the pipe network.

The program does not take into account the dry weather flow. If information and measurements are available from the urban area under consideration, the dry weather flow simply can be taken as an additional input for the routing through the sewers.

The data should be inserted according to the instructions provided in the appendices. However, the way of numbering the sewer system inlets should be further clarified. Starting from the remotest inlet, the inlets are numbered 1, 2, 3, etc. proceeding downstream until a junction is met. The junction is not numbered at that time but only after the new branch has been numbered with numbers of increasing order proceeding downstream. Figure 5-8 presents an example of the way of numbering inlets and gutters necessary for the computer input.

A listing of the UCUR Model Computer Program is included in the Appendix along with information on the preparation of the data cards.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INLETS
SEWER PIPES
GUTTERS

FIGURE 5-8. NUMBERING SEWER INLETS AND GUTTERS

UCUR MODEL LIMITATIONS

Although the UCUR Model is intended to be comparatively simple and easy to use, several limitations should be pointed out with regard to the basis on which it has been developed. Some of these will be discussed briefly here.

The model subtracts the calculated values of infiltration and depression storage supply from the rainfall intensity at any time to obtain the rainfall excess which produces overland flow. Thus, infiltration cannot be satisfied by rain water temporarily stored in depressions of the ground surface. Also the model assigns the depression storage full when the rainfall intensity falls below the infiltration capacity for the first time. Since depression storage reduces significantly the overland flow, especially from pervious areas, more research is needed to determine the proper depression storage to be used in the Model. Other factors reducing the overland flow which can be considered in the future are: evaporation, interception, and wetting of surfaces.

Further research is recommended concerning the capacity of the stormwater gutter inlets, because during heavy rainstorms there exists a carry-over flow (i.e. the portion of the gutter flow that does not go into an inlet).

If the water table of an urban basin is high, then groundwater infiltration or seepage through pipe joints, broken pipes, cracks, or openings in manholes and similar faults, should be taken into account. Research could be oriented to find a relation between pipe material, age of the sewer system, and groundwater infiltration or seepage.

Outlet conditions for direct application of the Model should be unsubmerged. If submergence exists, the Sewer Routing Sub-Model must be modified to reflect the backwater profile. Hence, more exact storage routing procedures should be used to route the flow through the laterals and sewer main. However, each improvement of the UCUR Model has to be justified by the additional manpower and computer time involved. Minimization of this effort time has been a prime objective in the UCUR Model development.

It should also be pointed out that the UCUR Model does not include provisions for reducing sewer system pipe sizes due to the availability of storage within the piping system or in intermediate ponding areas. It is expected that these additions can be made by modifying the Model to suit local conditions. In analyzing an existing system or in designing a new system, it is important to consider the extent of surface water ponding to be allowed. Therefore, the aspect of storage becomes a serious consideration with regard to sewer surcharging and flood damage.

CONCLUSIONS

A deterministic mathematical model known as the University of Cincinnati Urban Runoff (UCUR) Model has been presented as a practical method for calculating runoff hydrographs at desired points within an urban watershed. The procedure requires knowledge of the physical characteristics of the watershed, its existing drainage system, and local hydrology.

The UCUR Model is derived from the sequential addition of Sub-Models simulating separately the processes which control the rate of runoff. These include infiltration, surface retention, overland flow, gutter flow, and finally routing through the sewers.

Because of the extent of the computations involved, application of the UCUR Model can be carried out only by using a computer program such as the one developed at the University of Cincinnati which is presented in the Appendix along with detailed instructions for step by step preparation of the necessary data.

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SECTION 6

GENERAL DESCRIPTION OF EPA STORM WATER MANAGEMENT MODEL

SYNOPSIS

A number of mathematical models for the simulation of hydrologic runoff have been developed for various types of watersheds and may be found in the literature. More recently, some of these models have concentrated on urban runoff because of increasing drainage problems in urban areas. Compounding the problem of hydrologic drainage is the runoff and discharge of pollutants from combined sewers into receiving waters of various types. A number of studies have shown that the discharge of these pollutants from combined sewer systems may cause serious water pollution problems. Therefore, the Environmental Protection Agency contracted with a consortium of contractors for the development of a comprehensive mathematical model capable of representing urban stormwater runoff and combined sewer overflow phenomena. This model is known as the EPA Storm Water Management Model (SWMM) and was developed by the consortium including Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. The model has been published by EPA under the following titles and report numbers.

Title	EPA Report No.
STORM WATER MANAGEMENT MODEL Volume I - Final Report	11024 DOC 07/71
STORM WATER MANAGEMENT MODEL Volume II - Verification & Testing	11024 DOC 08/71
STORM WATER MANAGEMENT MODEL Volume III - User's Manual	11024 DOC 09/71
STORM WATER MANAGEMENT MODEL Volume IV - Program Listing	11024 DOC 10/71

Capabilities of the Model

1. The Storm Water Management Model accepts any rainfall hyetograph or multiple hyetographs (where several local rain gages are recording the same storm event) and produces a runoff hydrograph for each modeled watershed.

2. Continuous runoff quality graphs, called pollutographs, are computed on the basis of the volume of storm runoff and on antecedent conditions which include rainfall history, street-sweeping data, land use, and related data.

3. Hydrographs and pollutographs are computed for dry weather flows (in the case of combined systems) with daily and hourly variations and for infiltration.

4. Real time flow routing is accomplished through a simulation of the physical collection system allowing for variations in size, length, slope, and cross-section configuration in the case of conduits and locations and design parameters for non-conduits.

5. User options for storage facilities may be internal (in-system) or external (located at the overflow point). Options for storage are:

    a. Surface storage

    b. Intrasystem storage

    c. Flow rerouting

    d. Underground storage

    e. Underwater storage.

6. User options for treatment may be called in conjunction with or independent of external storage. The following treatment options and acceptable combinations of these options are provided:

    a. Mechanically cleaned bar racks

    b. Fine screens

    c. Sedimentation

    d. Dissolved air flotation

    e. Microstrainers

    f. High rate filters

    g. Effluent screens

    h. Chlorination

7. Receiving water effects in both flow and quality are computed for simulated lakes, rivers, or estuaries as applicable.

8. Capital costs and operation and maintenance costs are computed for the options selected to permit the devel-

opment of approximate cost-effectiveness curves which can be used for decision making.

Applications of the Model

1. The Storm Water Management Model is written in FORTRAN programming language with a minimum of machine-dependent features.

2. The computer hardware system should be the equivalent of the IBM 360/65 with peripheral storage devices and a usable core capacity of not less than 350K bytes.

3. The Model is developed on a general basis so it may be applied to any municipality by changing only the input data. The approximate size range of drainage basins over which the Model is applicable is from 10 acres to 5,000 acres.

4. The "User's Manual," Volume III, describes the main computer program, subroutines, data input and output, and instructions on applications to individual systems.

5. The user should be knowledgeable in FORTRAN programming, operating systems interfacing, and the engineering aspects of the real systems.

6. Data requirements are common to engineering design and analysis and are mainly descriptive of the real systems.

SYNOPSIS OF MODEL

Because of the length of the EPA-SWMM only a synopsis of the model follows so that the reader will have a basic understanding of it and how it may be used.

Section 7 of this report provides information and results from testing and comparing the model using actual field data collected for this purpose.

The Storm Water Management Model is designed to evaluate phenomena associated with stormwater runoff and sewage flows in combined sewer watersheds. It handles precipitation input to the drainage area and adds to the precipitation the pollutants accumulated as the water runs off the land (over building surfaces and ground) and comes into contact with waste discharges.

Dry weather flows and stormwater runoff comprise the two flows which the model analyzes in combination. The dry weather flow is the domestic and industrial wastewater of a community discharged into the conveyance system. It is diluted by infiltration and groundwater leakage. Its volume is relatively small but it is high in concentration of pollutants. The stormwater runoff is the precipitation during a storm minus the infiltration capacity and the ponding of the surface. This runoff is conveyed through gutters or pipes until it reaches the sewage system and combines with the sewage flow.

This combined flow is analyzed in the Model and is broken down into basic blocks or categories of which it is comprised. These five blocks are:

Surface runoff

Dry weather flow quantity

Infiltration

Transport

Storage

Each of these categories is described in general terms below.

Surface Runoff

Since the Model is designed for use anywhere in the United States it must be based solely on the information that can reasonably be expected to be available in any region to which it is applied. The information that should be available, then, is physical characteristics of the watershed and climatological data.

Subsequently the drainage basin is represented as an aggregate of idealized subcatchment areas. These subcatchments drain into "inlet manholes" or "gutter/pipes" which in turn convey the runoff to the main sewer system for routing. Hence, there are three types of routing elements: subcatchments (overland flow); gutters (channel flow); and pipes (special case of channel flow).

The Model accepts a hyetograph from the input rainfall data which is supplied by one or more raingage installations or perhaps a hypothetical or design rainstorm and produces a step-by-step accounting of the runoff through the three element types previously mentioned. Infiltration losses in pervious areas, surface detention, overland flow, gutter flow and contaminants washed into the inlets are all considered by the Model. From these the inlet hydrographs and pollutographs (BOD vs.Time and Suspended Solids vs. Time) are calculated.

A step by step computation is then made as indicated below for each subcatchment singly at first and then collectively as the runoff continues.

Step 1: Initially rainfall is added to the subcatchment from the input hydrograph:

$$D_1 = D_t + R_t \Delta t$$

where D_1 = Water depth after rainfall for time increment Δt

D_t = Water depth at time t, before Δt

R_t = Average intensity during time increment

Step 2: Infiltration is computed by Horton's relationship:

$$f = f_o + (f_i - f_o)\, e^{-kt}$$

where f = Infiltration capacity rate

t = Time from beginning of the infiltration capacity curve

f_o = Constant rate which f is asymptotically approaching as time continues

f_i = Infiltration capacity when $t = 0$

k = Decay rate of infiltration

Then:

$$D_2 = D_1 - f\ \Delta t$$

where D_2 = Water depth after infiltration is considered

Step 3: When the resulting water depth of the subcatchment is greater than the amount of surface detention the outflow is determined by Manning's equation. The overland flow depth is assumed uniform and is set equal to the hydraulic radius.

$$Q = \frac{1.49}{n} W(D_2 - D_d)^{5/3} S^{1/2}$$

where Q = Outflow rate

n = Manning's coefficient

S = Ground slope

W = Average width of subcatchment

D_d = Surface detention

Step 4: The continuity equation is used to compute the resulting depth using the following form.

$$D_{t+\Delta t} = D_2 - (Q/A)\ \Delta t$$

where A is the surface area of the subcatchment. This approach is a quasi steady state. The flow is assumed steady within each time interval Δt and the slope of the hydraulic grade line is set equal to the slope of the overland flow plane.

Step 5: The preceding four steps are repeated throughout the storm for each subcatchment.

Step 6: Inflow to the gutters is computed. This is a summation of the outflow from the subcatchments and the gutter flow immediately upstream. It is assumed that each gutter is fed uniformly along its length.

$$Q_{in} = \Sigma Q_{w,i} + \Sigma Q_{g,i}$$

where Q_{in} = Inflow to a gutter

$Q_{w,i}$ = Outflow from adjacent subcatchments

$Q_{g,i}$ = Flow from upstream gutter

Step 7: The water depth is added according to the inflow.

$$Y_1 = Y_t + (Q_{in}/A_s)\ \Delta t$$

where Y_1 and Y_t = Water depth in gutter

A_s = Mean of water surface area between Y_1 and Y_t

Step 8: Manning's equation for turbulent uniform flow is used for calculation of outflow from the gutter

$$Q_g = \frac{1.49}{n} A_c R^{2/3} S_i^{1/2}$$

where R = Hydraulic radius

S_i = Invert slope

A_c = Cross sectional area of gutter at Y_1

Step 9: The water depth of the gutters is computed by using the continuity equation.

$$Y_{t+\Delta t} = Y_1 + (Q_{in} - Q_g)\ \Delta t/A_s$$

The above steps are repeated until the flows in all the gutters subside. All the gutter flows are summed as they join together and hydrographs can be obtained at the sewer inlets. This step by step procedure constitutes the operation of the model for the hydrologic runoff simulation.

Dry Weather Flow

Dry weather flow characteristics are important to this Model since they occur before and during runoff from storms in combined sewers. Infiltration, although a part of the dry weather flow, will be considered in another section.

The Model is designed to accurately predict average wastewater flows taking into consideration daily and hourly fluctuations of domestic waste and flow quantities from industrial process flows. Availability is a limiting factor in making such predictions. Census tract and block statistical data as well as sewage flow and metered water use are the most complete and accurate accessible information. Total population, population density, land use, residential income, home valuation and other available factors are also of importance.

The Model has been designed to make estimates of wastewater inputs at certain locations along the trunk sewer of the drainage basin. These sewage inputs are added to infiltration estimates from all pipe sections and then added to runoff flows (when considering a combined sewage system). Hydrographs at certain locations throughout the sewage system can then be produced.

Equations representing residential water usage are based on the work done by Linaweaver [1]. Upon analyzing water use throughout the country, he stated that the principal factors influencing residential water use are primarily the number of homes along with other major factors - economic level of the consumer, climate, and whether consumers are metered or on a flat rate basis.

Water use data from 29 areas were analyzed by Linaweaver [1] for the development of the following equation:

$$Q_{d/a} = 178 + 3.28v$$

where $Q_{d/a}$ = Average domestic use per dwelling

v = Average assessed valuation of property (thousands of dollars)

Later analysis of residential water demand by Linaweaver and Howe [2] resulted in another equation.

$$q_{a,d} = f(v,a,d_p,k,p_w)$$

where $q_{a,d}$ = Annual average quantity demanded for domestic purposes in gallons per dwelling unit per day (gpd/dwelling)

v = market value of dwelling (thousands of dollars)

a = age of dwelling (years)

d_p = number of persons per dwelling unit

k = average water pressure (psi.)

p_w = sum of water and sewage charges that vary with water use, evaluated at the block rate applicable to the average domestic use in each drainage basin

From an analysis of 21 drainage areas, the following relationships were developed.

Metered with public sewer:

$$q_{a,d} = 206 + 3.47v - 1.30p_w$$

Flat rate and apartments with public sewer:

$$q_{a,d} = 28.9 + 4.39v + 33.6d_p$$

Metered with septic tank

$$q_{a,d} = 30.2 + 39.5d_p$$

In estimating the flow quantity, three general data categories are used for estimation. They are drainage basin data, subarea data, and decision and adjustment parameters.

The first category simply includes the number of subareas into which the drainage basin is divided and the total surface area of the drainage basin.

The second category deals with identification parameters, flow data and estimating data. The identification parameters include the number of manholes into which the dry weather flow is assumed to enter and the zoning classifications for each subarea. These zoning classifications are given in Table 4-2, Volume III of the Storm Water Management Model. If flow data are known, which usually is not the case, the predictive equations of the previous section are not needed. These flow data could be obtained from average sewage flow measurements or metered water use. However, when these flow data are not available, data estimations must be made for water use. For industrial or commercial areas estimates can be made by multiplying an established coefficient by the number of units on which the coefficient is based. These can be found in Appendix A, Volume III of the Storm Water Management Model. These values are then summed together for each subarea. When considering residential areas, the factors described and used by the Linaweaver and Howe relationships may be used.

Decision and adjustment parameters comprise the third category. These are basically factors which control average daily and weekly flow fluctuations and adjust water prices to a 1963 Scale. These are the scales used in the development of the Model. Local observations are better for time variations of flows but a general representation is given for model usage. The consumer price index was chosen to adjust water prices and the composite construction cost index was chosen to adjust house valuations.

Sewer Infiltration

Infiltration of ground water into sewers, such as through leaky joints, is of prime concern when considering basic sewer flow without storm water runoff. Therefore, it has been incorporated into the Model as an addition to the dry weather flow criteria discussed previously. Since actual sewer infiltration may vary considerably with the sewer system, representative data from a residential sewer system research project by Johns Hopkins University were used. In addition, existing information on a sewer system, data on soil and groundwater, and climatological data are used to predict infiltration additions at certain locations along the trunk sewers.

When the groundwater table is above the invert of the sewer pipe, the total infiltration can be obtained from historical sewer flow data by inspection or regression analysis. Regression analysis would require the following equation.

$$\mathrm{GINFIL} = \Sigma_{i=1}^{n} (\mathrm{BETA}(i) \quad (\mathrm{GWHD})^{\alpha_i})$$

where GINFIL = Groundwater infiltration

GWHD = Groundwater table elevation above invert of sewer pipe

BETA(i) = Coefficient for the ith term of the equation

α_i = Power to which GWHD is raised in the ith term, e.g., 0,1,2,1/2, for term i = 0,1,2, 3 respectively

If, however, the groundwater level is below the sewer invert the infiltration could be a summation of three factors as follows.

$$QINF = DINFIL + RINFIL + SINFIL$$

where QINF = Total infiltration

DINFIL = Dry weather infiltration

RINFIL = Wet weather infiltration - DINFIL

SINFIL = Melting residual ice and snow infiltration

RINFIL is dependent upon the antecedent rainfall and is described as:

$$RINFIL = ALF + (ALF_0)(RN_0) + \ldots\ldots\ldots (ALF_9)(RN_9)$$

where RN_n = Precipitation on n days prior to estimate

ALF_n = Coefficient to rainfall for n days prior to estimate

These coefficients are obtained by running a linear regression on existing flow and rainfall data. In Table 4-1, Volume III of the Storm Water Management Model coefficients similarly obtained are given for three cities for the sake of comparison.

Base dry weather sewer infiltration should be obtained from actual flow data when available. Often, however, such data are not available and it is necessary to estimate the unit infiltration rate for addition to the dry weather flow. From this estimate, DINFIL is obtained.

$$DINFIL = (XLOCAL)(DIAM)(PLEN)$$

where XLOCAL = Estimate obtained from local sources

DIAM = Average sewer diameter

PLEN = Pipe length

Infiltration from the melting of residual ice and snow can be obtained from the following relationship if the day under study is within the melting period for the study area.

$$SINFIL = RSMAX(Sin[180\frac{NDYUD-MLTBE}{MLTEN-MLTBE}])$$

where RSMAX = Residual moisture peak contribution

NDYUD = Day on which estimate is desired

MLTEN = Day on which melting period ends

MLTBE - Day on which melting period begins.

Transport

The purpose of the transport section is to accept the inputs into the sewer system from the surface runoff section of the Model and properly represent the flow in the storm or combined sewers.

The theoretical transport process (ref. pg. 117, Volume I of the Storm Water Management Model) has been developed from the simultaneous solution of the following two equations:

$$\Psi = \frac{AR^{2/3}}{A_f R_f^{2/3}} \text{ and}$$

$$\Psi_{j+1,n+1} + C_1 \alpha_{j+1,n+1} + C_2 = 0$$

where $\alpha = A/A_f$ and $\Psi = Q/Q_f$

Subscript f denotes conditions of the conduit flowing full. Area, hydraulic radius and discharge are indicated by A, R and Q respectively. The coefficients C_1 and C_2 deal with time and spatial contributions of the flow. The above two relationships emanated from the continuity equation and Manning's equation using the slope of the energy grade line. However, because the flow is considered steady within each time interval, the accuracy of the simulation would decrease with increasing time intervals.

Storage

The storage section of the Model has been developed to account for the effects of storage elements as they hold back or slow down the flows through a sewer system. Storage elements serve, primarily, a two-fold purpose. They may reduce the peak flow rate or they may reduce the amount of pollutants discharged during the storm, especially during the initial flush.

Besides aiding the prediction of runoff, this section of the Model also is quite useful to an engineer designing needed storage elements in a sewer system. The engineer could input a hypothetical storage element, observe the outcome, then make adjustments on the element until the desired effect is attained.

Development of the storage Model was based on the basic storage equation (outflow = inflow + or - change in storage). This storage is divided into two basic types --- internal and external storage. Internal storage deals with small storage tanks located on major upstream branches of the sewer system while external storage deals with major installations at the sewer system outfall.

As internal storage inflows become known all transport element flows, such as these storage elements, can be solved in a step by step process. Each element has a maximum storage capacity which, when exceeded, is said to be flooded out and its modeling discontinued. External storage is usually located at sewer outfalls and is generally a major installation. Its capacity is computed in a similar manner as the internal storage.

The basic routing equation is as follows:

$$0.5D_t + S_2 = 0.5(I_1+I_2)D_t - (0.5\ D_tO_1 - S_1)$$

where I = Inflow rate

O = Outflow rate

S = Stored volume

D_t = Hydrograph time-step size

and 1 = conditions at the beginning of D_t

2 = conditions at the end of D_t

Where two types of outlets are considered:

Orifice: $Q_{out} = C_dA(64.4)^{1/2}$

Weir: $Q_{out} = 3.33\ LH^{1.5}$

where C_d = Coefficient of discharge

A = Orifice area

D = Water depth above orifice centerline

L = Length of weir

H = Head over weir crest

In the computation of storage quantities, the physical characteristics of the storage elements are required. There are two basically different types of storage. They are the irregular or natural reservoirs and the man-made units. For a man-made unit, dimensions must be given so that the basic routing equation can be used. For an irregular or natural reservoir, 11 pairs of depth versus surface area data must be provided by the user.

The definite distinction between these five major computational categories and the five program blocks should be noted. These categories were chosen in order to organize the discussion of the theoretical and mathematical bases of the Model. The program blocks (Executive, Runoff, Transport, Storage and Receiving Water) form the basic structure of the computer program itself and are not specifically oriented toward theory.

INPUT DATA

Data input consists of three main steps. The first step is data preparation for the surface runoff section. This includes one or more rainfall hyetographs, Horton's infiltration parameters, depression storage values, impervious area with zero depression storage, and subcatchment characteristics (i.e. area, slope, width, roughness).

The second step consists of the data describing the sewer system. These data consist of element types and their physical description throughout the drainage area. The system is described by conduit elements joined by non-conduit elements. The shape, cross-section dimensions, length, slope and roughness of each conduit are required. Flow-area relationships are included in the Model for twelve conduit shapes (i.e. circular, rectangular, trapezoidal,rectangular with triangular bottom, etc.) Description of these twelve conduit shapes and the necessary parameters as well as information on non-conduit elements are included in Volume III (pg. 153-162) of the Storm Water

Management Model.

Step three is simply inputting basic data, such as hydrographs and pollutographs already generated by the runoff section.

SUMMARY

The five major sections described above comprise the theoretical bases of the simulation techniques employed by the Storm Water Management Model.

The Model is written in FORTRAN programming language and is compatible with a computer hardware system equivalent to the IBM 360/65 with peripheral storage devices and a usable core capacity of not less than 350K bytes.

Application of the model may be made to a range of drainage basins generally within a size of from 10 to 5,000 acres. It may be applied to any municipality by inputting the appropriate data.

REFERENCES

Section 6

1. Linaweaver, F.P., "Final and Summary Report on the Residential Water Use Research Project", July 1966, John Hopkins University, Baltimore, Maryland.

2. Howe, C.W. and Linaweaver, F.P., "The Impact of Price on Residential Water Demand and Its Relation to System Design and Price Structure", Water Resources Research, Vol. 3, No. 1, 1967.

SECTION 7

VERIFICATION AND TESTING OF THE EPA STORM WATER MANAGEMENT MODEL

For verification and testing of the EPA Storm Water Management Model, data have been collected over a three-year period from the Bloody Run Sewer Watershed in Cincinnati, Ohio. This is a combined sewer urban watershed of approximately 2380 acres of rolling terrain in an area typical of the east central part of the United States. A summary of the physical characteristics of the watershed is given below; however, for further details and data, the reader is referred to Volume II of this report.

GENERAL PHYSICAL CHARACTERISTICS OF BLOODY RUN SYSTEM

The watershed is referred to as Bloody Run because of Indian massacres of white settlers traveling through that area during an early historical period. Its location is in the northeastern section of Cincinnati as shown in Figure 7-1. Topographically, the area is characterized by two shallow valleys running approximately east and west. Most of the commercial and industrial sections are located in these valleys; the residential housing is found on the ridges. Figure 7-2 shows an aerial photo of the watershed. About 55% of the area is residential, 17% commercial, 5% industrial, and 22% open land and parks. A more detailed division of the Bloody Run Sewer Watershed into different land uses is presented in Figure 7-3 and Table 7-1.

The Bloody Run Sewer Watershed was divided into 38 subcatchments by using sewer, topographical and zoning maps, with the concept that each of these areas should include one major type of land use and should incorporate an individual inlet manhole. The resulting division is shown in Figure 7-4. The subcatchments have been numbered in such a way as to follow the sewer network and facilitate the data tabulation for computer programming.

The physical characteristics of the subcatchments, i.e. their area, length, average width (which is used to calculate the distance that the overland runoff must travel before entering the inlet manhole), maximum and minimum elevation, average slope, and perviousness, are tabulated in Tables 7-2 and 7-3.

The total population according to the 1960 census is approximately 26,000 with an average of about 11 persons per acre. More detailed information is listed in Table 7-4.

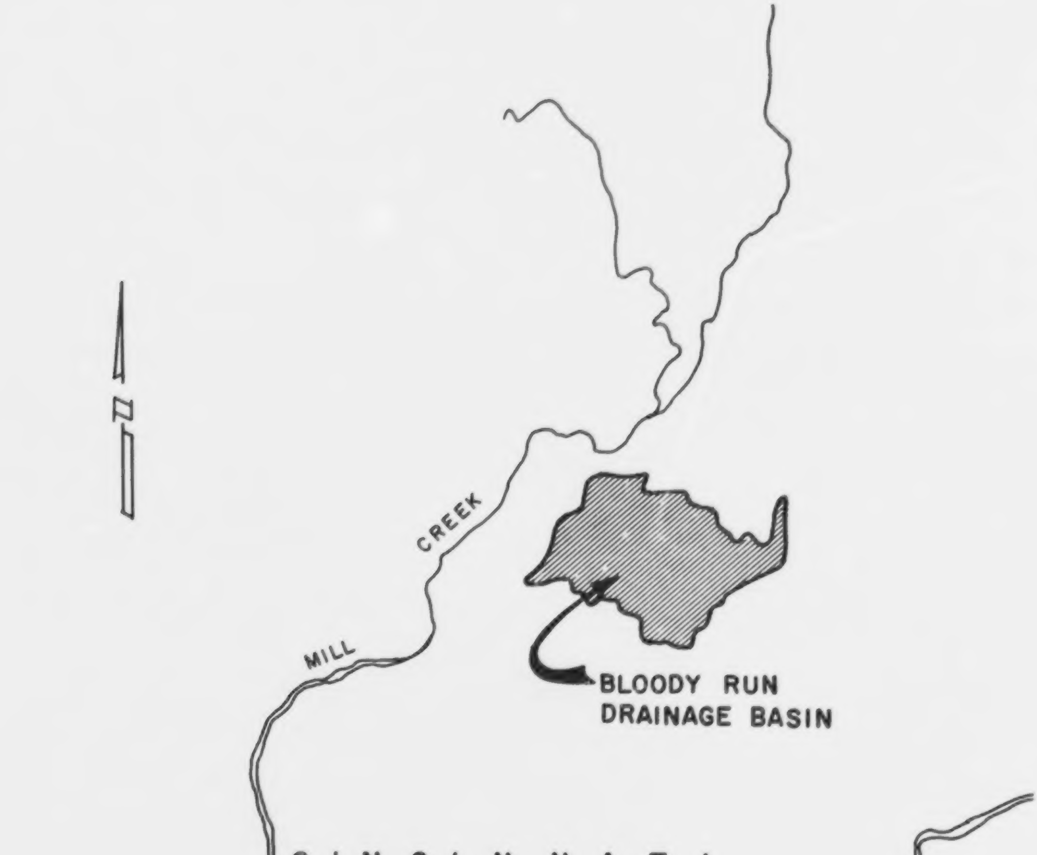

N
CREEK
MILL
BLOODY RUN
DRAINAGE BASIN
CINCINNATI
OHIO
RIVER

Figure 7-1. GENERAL LOCATION MAP OF CINCINNATI BLOODY RUN DRAINAGE BASIN.

Reproduced from
best available copy.

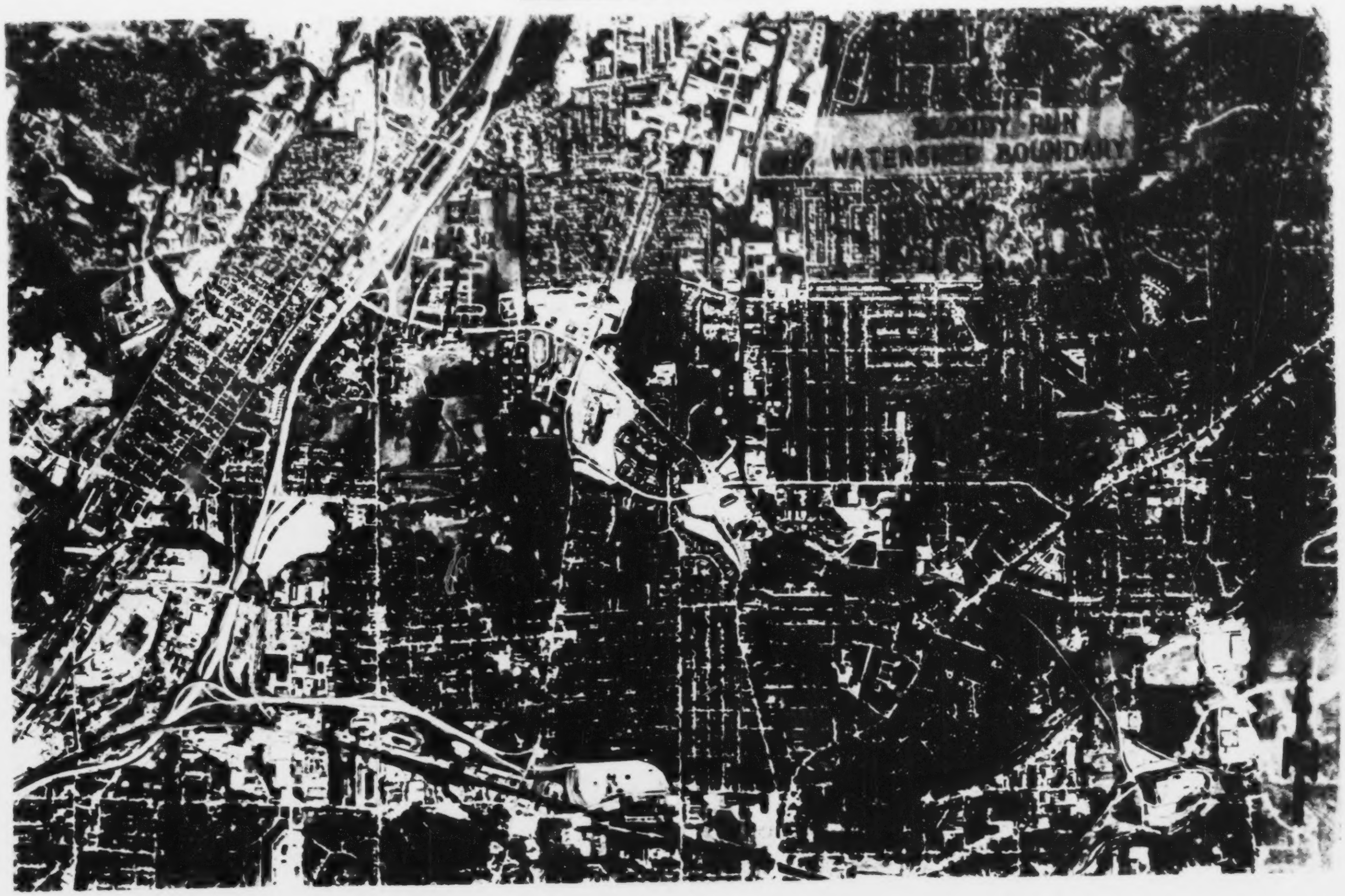

BLOODY RUN
WATERSHED BOUNDARY

Figure 7-2. AERIAL PHOTOGRAPH OF THE BLOODY RUN SEWER WATERSHED.

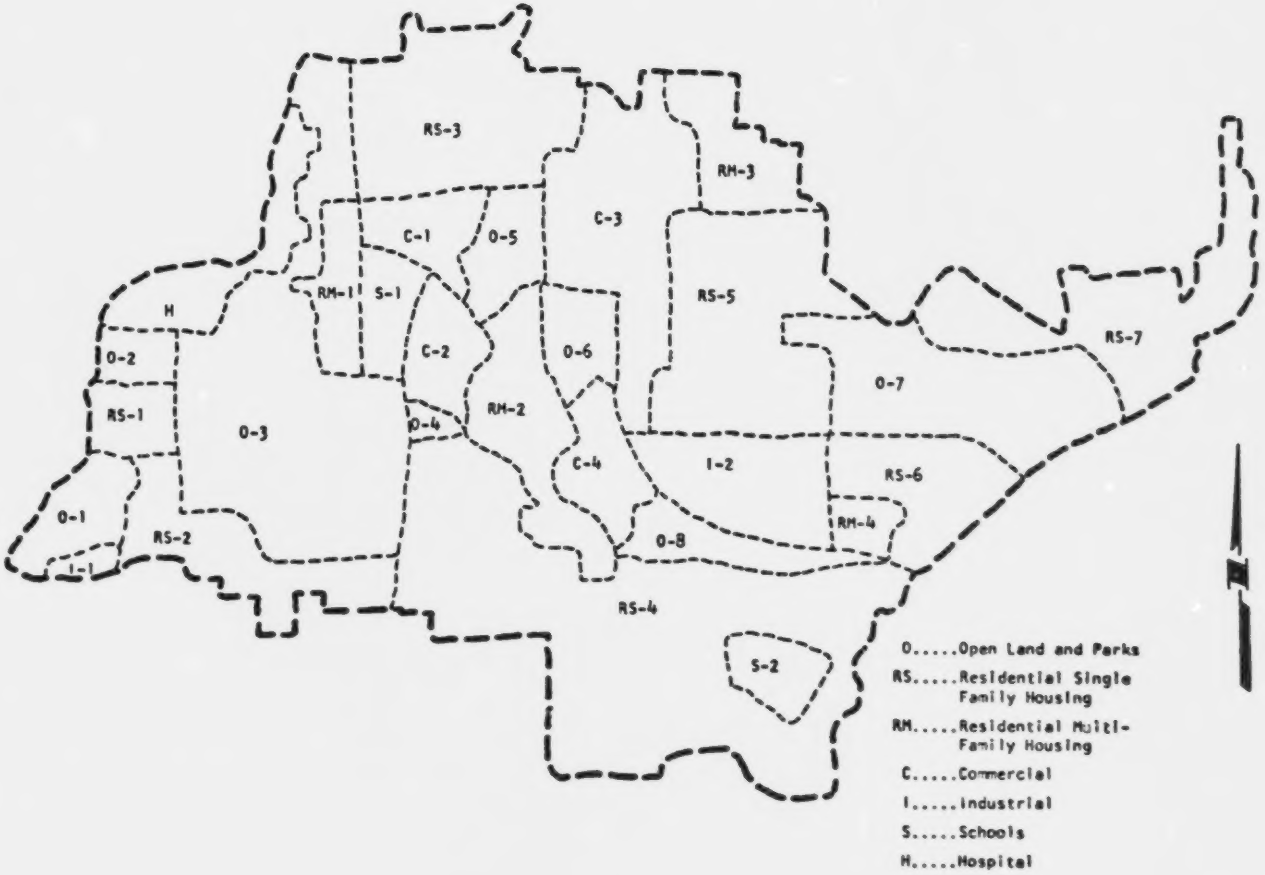

RS-3
RM-3
C-1
O-5
C-3
RM-1
S-1
RS-5
H
C-2
O-6
RS-7
O-2
O-7
RS-1
O-4
RM-2
O-3
C-4
I-2
RS-6
O-1
RM-4
RS-2
O-8
I-1
RS-4
S-2
O.....Open Land and Parks
RS.....Residential Single Family Housing
RM.....Residential Multi-Family Housing
C.....Commercial
I.....Industrial
S.....Schools
H.....Hospital

Figure 7-3. DIVISION OF BLOODY RUN SEWER WATERSHED INTO DIFFERENT LAND USES.

Table 7-1. DIVISION OF BLOODY RUN SEWER WATERSHED INTO DIFFERENT LAND USES

Char.*	Area (acres)	Land Use
0-1	38.5	Open Land and Parks
0-2	18.8	
0-3	275.0	
0-4	9.2	
0-5	34.5	
0-6	34.5	
0-7	138.0	
0-8	32.2	
RS-1	27.6	Residential Single Family Housing
RS-2	113.0	
RS-3	160.0	
RS-4	396.0	
RS-5	216.0	
RS-6	78.0	
RS-7	137.5	
RM-1	27.6	Residential Multi-Family Housing
RM-2	85.5	
RM-3	57.5	
RM-4	11.5	
C-1	39.2	Commercial
C-2	38.0	
C-3	160.0	
C-4	46.0	
I-1	9.2	Industrial
I-2	79.5	
S-1	34.5	Schools
S-2	40.0	
H	39.0	Hospital

*Referring to Figure 7-3.

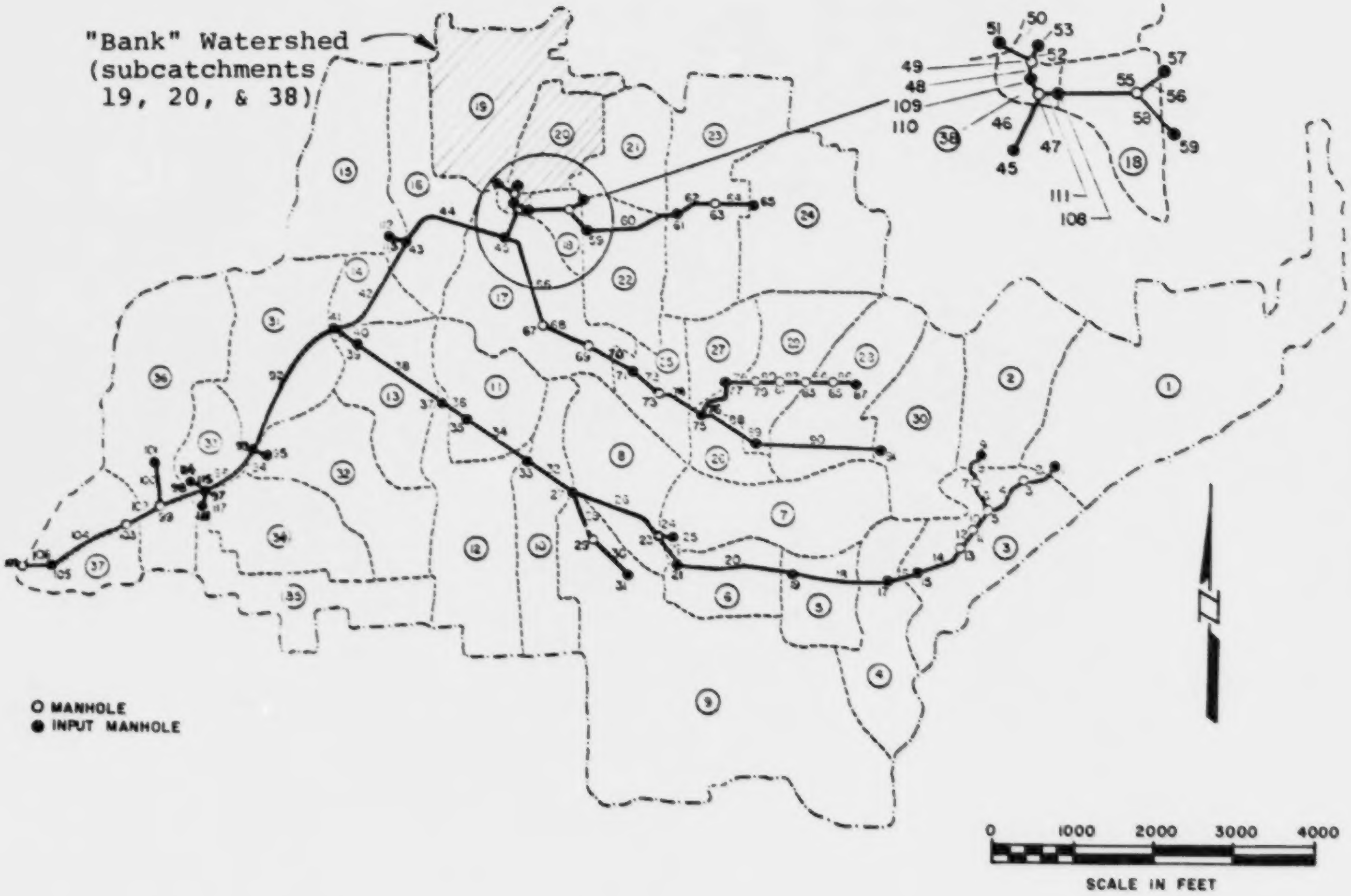

"Bank" Watershed
(subcatchments
19, 20, & 38)
51
50
53
52
49
48
109
110
46
47
45
38
55
57
56
58
59
18
111
108
O MANHOLE
● INPUT MANHOLE
0
1000
2000
3000
4000
SCALE IN FEET

Figure 7-4. PLAN OF CINCINNATI BLOODY RUN SYSTEM.

Table 7-2. PHYSICAL CHARACTERISTICS OF THE BLOODY RUN WATERSHED.

Subcatchment Number	Length in Feet (L)	Average Width in Feet (A/L)	Area in Acres (A)	Perviousness (%)
1	5010.0	1528.1	176.0	66.3
2	2625.0	1136.8	68.6	71.3
3	2526.5	1260.3	73.2	46.5
4	625.1	4105.7	59.0	65.3
5	1062.5	1555.8	38.0	63.2
6	1500.0	968.6	33.4	61.1
7	3062.5	980.1	69.0	26.7
8	1375.0	1698.9	53.7	43.2
9	4562.5	2385.5	250.2	48.1
10	750.0	2621.6	45.2	41.5
11	1125.0	1484.8	38.4	18.6
12	1375.0	2587.8	81.8	45.5
13	1375.0	1575.5	49.8	89.2
14	2312.5	380.0	20.2	65.3
15	2687.5	1165.4	72.0	79.3
16	1437.5	3510.3	116.0	54.4
17	2500.0	1586.9	91.2	35.5
18	1125.0	692.1	17.9	75.5
19	1250.0	2049.7	58.9	60.2
20	1125.0	1643.3	42.5	60.3
21	875.0	1794.7	36.1	57.7
22	1125.0	1542.8	39.9	53.6
23	1062.5	3566.0	87.1	39.2
24	1562.5	2458.3	88.3	61.3
25	1562.5	838.0	30.1	55.4
26	1000.0	1300.6	29.9	28.3
27	750.0	1270.2	21.9	43.6
28	1125.0	2331.6	60.3	51.3
29	1000.0	2088.0	48.0	60.1
30	1125.0	1956.5	50.6	70.3
31	2312.5	1241.5	66.0	95.0
32	2600.0	903.5	54.0	95.1
33	1300.0	836.5	25.0	83.2
34	2625.0	830.2	50.1	89.1
35	3250.0	836.5	62.5	60.5
36	710.0	7474.6	122.0	50.3
37	875.0	2436.0	49.0	66.2

Table 7-3 COMPUTED AVERAGE SLOPE OF SUBCATCHMENTS OF BLOODY RUN SEWER WATERSHED.

Subcatchment Number	Maximum Elevation	Minimum Elevation	Distance (feet)	Slope (%)
1	845	690	5000	3.1
2	750	675	2400	3.1
	785	750	450	7.8
3	700	645	1800	3.1
	730	675	700	7.8
4	680	650	1300	2.3
	715	650	850	7.6
5	720	605	1050	11.0
6	685	625	600	10.0
7	650	600	2300	2.2
	675	650	500	5.0
	675	650	400	6.3
8	620	575	400	11.3
	620	575	600	7.5
9	800	625	4000	4.4
	715	625	2600	3.5
10	625	560	2400	2.7
11	585	555	700	4.3
12	610	555	1700	3.2
13	595	550	900	5.0
14	580	545	1000	3.5
15	600	560	1300	3.1
	590	560	700	4.3
16	595	540	1200	4.6
17	600	550	2100	2.4
18	590	550	1000	4.0
19	595	575	900	2.2
20	590	575	1000	1.5
21	575	555	600	3.3
22	600	575	1000	2.5
23	620	590	1400	2.1
24	725	605	1900	6.3
25	625	575	600	8.3
26	635	600	550	6.4
27	670	605	1100	5.9
28	725	640	2000	4.3
29	660	610	1400	3.6
30	735	660	1400	5.4
	730	660	900	7.8
31	590	540	1200	4.2
	580	540	700	5.7
32	600	535	2600	2.5
33	575	540	1300	2.7
34	575	535	1700	2.3
35	610	600	800	1.2
36	585	530	750	7.3
	575	540	600	5.8
37	575	525	700	7.1

Table 7-4. CENSUS DATA, YEAR 1960, FOR BLOODY RUN SEWER WATERSHED.

Subcatchment Number	Total Number of Persons per Acre	Total Number of Dwelling Units	Number of People Living in Avg. Dwelling Unit	Median Family Income ($1000)	Avg. Market Value Owner Occupied Units ($1000)
1	9.29	608	2.83	8.34	21.31
2	9.72	230	2.65	7.62	17.42
3	8.72	215	2.79	7.64	19.00
4	13.46	178	2.78	7.64	19.00
5	15.19	111	2.78	7.64	19.00
6	13.97	98	2.78	7.64	19.00
7	8.30	203	2.78	7.64	19.00
8	14.68	302	2.89	7.60	18.20
9	18.19	1620	2.63	7.38	17.30
10	17.09	307	2.63	7.38	17.30
11	16.74	223	2.75	7.47	17.38
12	12.57	395	2.71	7.29	17.08
13	8.39	105	3.91	8.34	20.91
14	10.48	40	4.53	8.61	22.70
15	10.48	140	4.53	8.61	22.70
16	10.48	100	4.53	8.61	22.70
17	12.54	335	3.90	8.16	20.85
18	10.48	40	4.53	8.61	22.70
19	10.48	140	4.53	8.61	22.70
20	5.86	130	4.53	8.61	22.70
21	10.48	50	4.14	12.72	31.30
22	6.67	110	4.53	8.61	22.70
23	6.73	192	3.26	11.21	26.32
24	10.98	321	3.12	8.33	19.23
25	11.01	91	3.89	8.23	20.49
26	9.22	58	2.79	7.39	17.81
27	11.26	81	3.02	7.53	17.30
28	11.62	231	3.15	7.55	17.43
29	11.26	179	3.02	7.53	17.30
30	13.09	57	3.70	7.58	18.54
31	10.48	124	4.53	8.61	22.70
32	5.38	100	2.88	7.24	17.00
33	7.93	47	3.37	7.21	18.07
34	5.38	93	2.88	7.24	17.00
35	5.38	116	2.88	7.24	17.00
36	8.38	266	3.99	8.24	20.95
37	10.48	113	4.53	8.61	22.70

The Bloody Run Watershed has been monitored at five stations along the pipe network. Namely the Bank 54", Bank 78", Longview 12' x 9', Longview 11' x 9', and Outlet Monitoring Stations. Their locations in the watershed are shown on Figure 7-5. These stations have been recording runoff over a 3-year period from 1970 to 1972 and samples have been taken for water quality analyses during stormwater runoff. Three raingages located throughout Bloody Run (Figure 7-5) have automatically recorded the accumulation of rainfall with respect to time. The hyetographs produced from data collected from these raingages were used as input to the Model. A subsection of approximately 105 acres in the Bloody Run Watershed was also selected for testing. This area, called the "Bank Watershed" in this report, drains only the subcatchments 19, 20, and 38 as indicated in Figure 7-4. Its outlet is the Bank 54" monitoring station. This smaller area was selected because of the greater discretization which could be accomplished relative to the larger drainage area, i.e., Bloody Run itself. Computer output could then be compared for the two different levels of discretization using results associated with the Bank 54" station. When modeled in its fullest detail, the Bank Watershed is composed of 95 subcatchments drained by "gutter/pipes". The sewer pipe network, shown on Figure 7-6, includes pipes as small as 8" in diameter. Figure 7-6 also illustrates the way of numbering the pipe elements. Information on sewer pipe diameters, lengths, slopes and roughness are presented in Table 7-5.

Initially, a check was made to be certain the system was not surcharged for the flows being considered. Therefore, capacities were computed and compared with maximum flows reported over the three years of monitoring. These peak flow rates were well below the capacity. Therefore, it was concluded that surcharging of the system could be neglected.

TESTING OF THE EPA STORM WATER MANAGEMENT MODEL

The data collected under the Cincinnati Urban Runoff Characteristics Project are in hydrograph and pollutograph form and were used to test the RUNOFF, TRANSPORT, and GRAPH blocks of the Storm Water Management Model. Data were not used for the testing of the STORAGE and RECEIVING WATER blocks. The recorded hydrographs, hyetographs, and pollutographs were those obtained from the Bloody Run Watershed operations as contained in Volume II, FIELD DATA.

More than 150 comparisons of computer output and measured data were carried out with the aid of the IBM 360/65 computer at the University of Cincinnati. These test comparisons are listed in Appendix A along with their associated input variations.

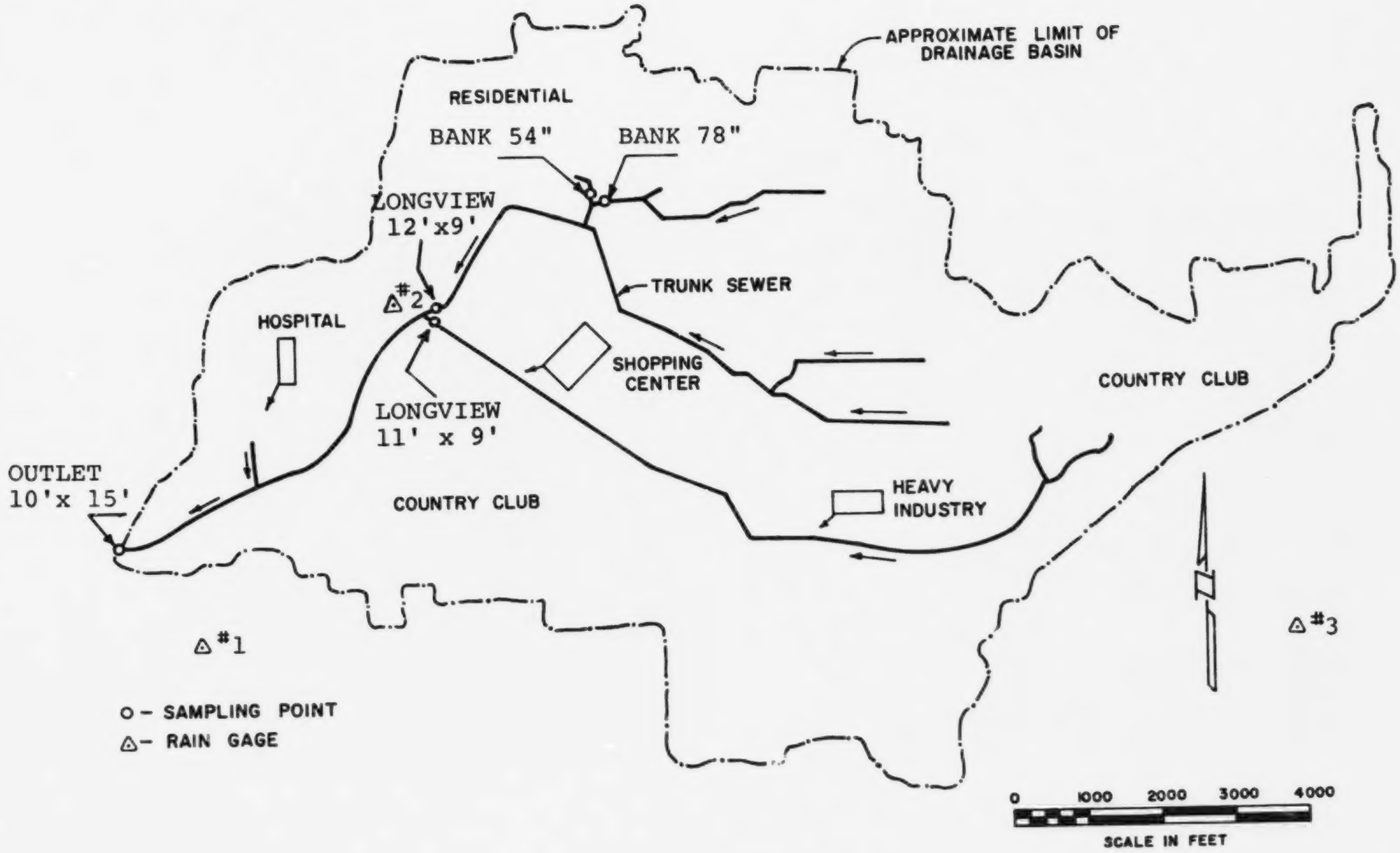
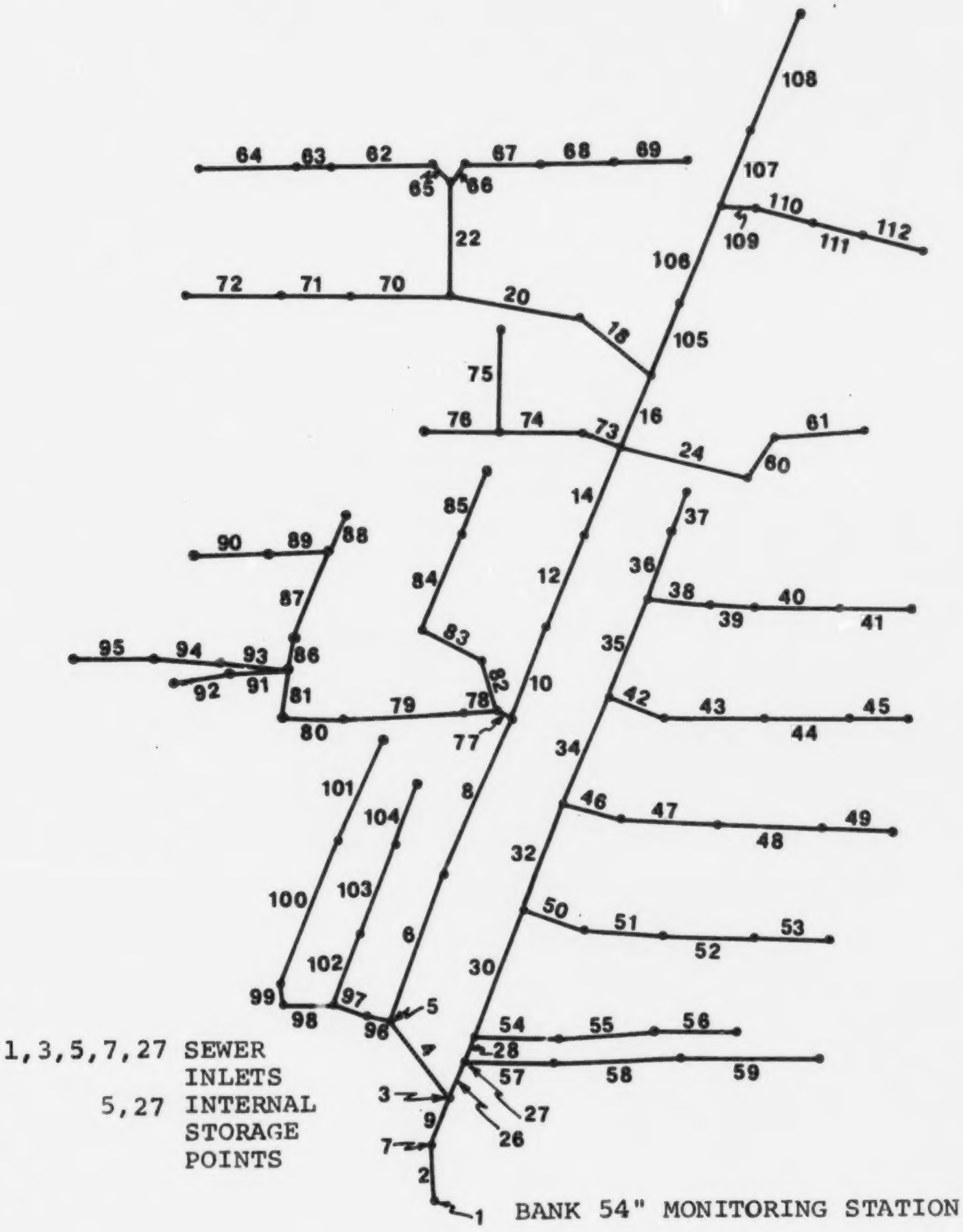

APPROXIMATE LIMIT OF DRAINAGE BASIN
RESIDENTIAL
BANK 54"
BANK 78"
LONGVIEW 12'x9'
$\triangle$#2
HOSPITAL
TRUNK SEWER
SHOPPING CENTER
LONGVIEW 11' x 9'
COUNTRY CLUB
OUTLET 10'x 15'
COUNTRY CLUB
HEAVY INDUSTRY
$\triangle$#1
$\triangle$#3
o - SAMPLING POINT
$\triangle$- RAIN GAGE
0
1000
2000
3000
4000
SCALE IN FEET

Figure 7-5. CINCINNATI RAINGAGE AND RUNOFF SAMPLING POINT LOCATIONS.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1,3,5,7,27 SEWER INLETS
5,27 INTERNAL STORAGE POINTS
BANK 54" MONITORING STATION

Figure 7-6. SEWER PIPE NETWORK FOR "BANK" WATERSHED AND NUMBERING OF PIPE ELEMENTS

Table 7-5. ELEMENTS INFORMATION "BANK" WATERSHED

Pipe No. *	Diameter (ft.)	Length (ft.)	Slope (ft/ft)	Manning's n
2	4.50	161	0.0138	0.015
9	4.50	161	0.0138	0.015
4	4.00	242	0.0209	0.015
26	3.00	61	0.0270	0.015
6	4.00	396	0.0092	0.012
8	4.00	431	0.0093	0.012
10	4.00	186	0.0075	0.012
12	4.00	339	0.0068	0.012
14	4.00	251	0.0076	0.012
16	3.50	202	0.0078	0.012
18	3.00	140	0.0033	0.012
20	3.00	200	0.0035	0.012
22	2.50	332	0.0050	0.012
24	2.50	368	0.0075	0.012
28	3.00	24	0.0356	0.012
30	2.50	405	0.0122	0.012
32	2.50	316	0.0101	0.012
34	2.00	278	0.0101	0.012
35	2.00	303	0.0081	0.012
36	2.00	302	0.0054	0.012
37	1.25	47	0.0244	0.012
38	1.50	158	0.0118	0.012
39	1.00	105	0.0110	0.012
40	1.00	251	0.0082	0.012
41	1.00	250	0.0070	0.012
42	1.50	155	0.0129	0.012
43	1.00	197	0.0123	0.012
44	1.00	250	0.0103	0.012
45	1.00	252	0.0063	0.012
46	1.50	155	0.0290	0.012
47	1.00	259	0.0122	0.012
48	1.00	259	0.0084	0.012
49	1.00	262	0.0062	0.012
50	1.50	157	0.0201	0.012
51	1.00	217	0.0157	0.012
52	1.00	217	0.0121	0.012
53	1.00	240	0.0072	0.012
54	1.00	241	0.0355	0.012
55	1.00	206	0.0301	0.012
56	1.00	225	0.0177	0.012
57	1.50	249	0.0347	0.012
58	1.50	349	0.0029	0.012
59	1.50	335	0.0021	0.012
60	1.25	144	0.0091	0.012

Table 7-5 (Continued)

Pipe No. *	Diameter (ft.)	Length (ft.)	Slope (ft/ft)	Manning's n
61	1.25	259	0.0060	0.012
62	1.75	276	0.0077	0.012
63	1.50	71	0.0078	0.012
64	0.67	233	0.0073	0.012
65	0.75	33	0.0082	0.012
66	1.50	45	0.0102	0.012
67	1.00	199	0.0042	0.012
68	1.00	202	0.0038	0.012
69	1.00	150	0.0048	0.012
70	1.50	275	0.0038	0.012
71	1.00	237	0.0052	0.012
72	1.00	249	0.0054	0.012
73	2.00	140	0.0380	0.012
74	1.00	273	0.0031	0.012
75	1.00	316	0.0030	0.012
76	0.67	278	0.0059	0.012
77	2.50	44	0.0184	0.012
78	2.50	100	0.0100	0.012
79	2.50	322	0.0184	0.012
80	2.50	120	0.0088	0.012
81	2.50	145	0.0120	0.012
82	1.25	162	0.0222	0.012
83	1.25	145	0.0207	0.012
84	1.00	274	0.0074	0.012
85	1.00	320	0.0096	0.012
86	2.00	55	0.0048	0.012
87	1.50	263	0.0092	0.012
88	1.50	60	0.0092	0.012
89	1.00	165	0.0100	0.012
90	1.00	166	0.0109	0.012
91	1.00	161	0.0181	0.012
92	1.00	160	0.0059	0.012
93	0.67	216	0.0119	0.012
94	0.67	201	0.0073	0.012
95	0.67	219	0.0079	0.012
96	1.75	30	0.0275	0.012
97	1.75	107	0.0032	0.012
98	1.75	105	0.0313	0.012
99	1.75	50	0.0340	0.012
100	1.25	369	0.0054	0.012
101	1.00	322	0.0055	0.012
102	0.67	107	0.0032	0.012
103	0.67	256	0.0047	0.012

Table 7-5 (Continued)

Pipe No. *	Diameter (ft.)	Length (ft.)	Slope (ft/ft)	Manning's n
104	0.67	217	0.0023	0.012
105	2.00	143	0.0086	0.012
106	2.00	265	0.0080	0.012
107	0.67	158	0.0043	0.012
108	0.67	315	0.0039	0.012
109	2.00	62	0.0101	0.012
110	1.50	133	0.0109	0.012
111	1.50	153	0.0086	0.012
112	1.00	60	0.0100	0.012

* Pipe numbers refer to Figure 7-6.

The following testing procedure was employed using the entire Bloody Run Watershed and the smaller Bank Watershed with greater discretization of the pipe network, as previously described.

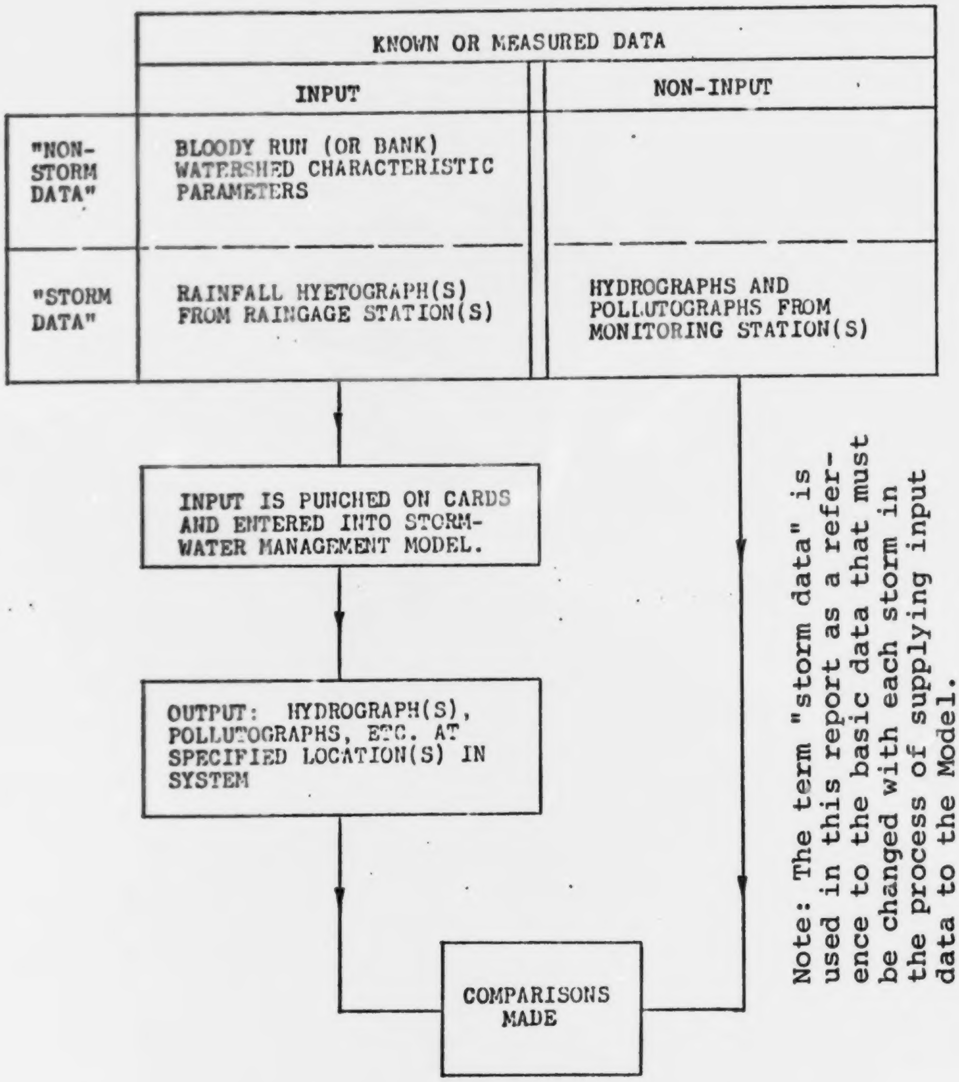
A range of storms were selected for testing so that they would be representative of varied conditions of runoff. These storms were represented in the form of hyetographs: three for the Bloody Run Watershed and one for the small Bank Watershed. In addition to the storm data, the system was characterized in the form of various parameters such as pipe network data, infiltration data, land use information, etc.

The preceding information was punched onto data cards prior to each run in accordance with instructions in the SWMM USER's MANUAL, Volume III. For each program block, the USER's MANUAL provides the following additional information:

1) Descriptions of the program subroutines with flow charts.
2) Instructions on data preparation with tables for data card input requirements and an alphabetical list of variables.
3) Examples of the application of procedures described with sample I/O information reproduced.

The data cards are then entered into the computer with the main program having been stored on tape previously. The output is in the form of hydrographs and pollutographs (BOD, SS, coliforms, etc) at specified locations in the system. Comparisons are then made between these computed hydrographs and pollutographs and the measured (field) hydrographs and pollutographs. Figure 7-7 provides a general flow diagram of the testing procedure described above.

In an effort to improve the fit between the measured and computed curves and also to examine the effects of various parameters on the output, certain modifications of the input data were made. These included changes in the infiltration data and the addition of "gutter-pipe" information to the RUNOFF block data. (Gutter/pipes are those pipes smaller than 2 to 3 feet in diameter with no backwater effects, flow dividers, or lift stations, and they are so designated for computation by the RUNOFF block of the model. These pipes are not connected to one another by manholes but join directly and lead to an inlet manhole for further routing (by the


KNOWN OR MEASURED DATA
INPUT
NON-INPUT
"NON-STORM DATA"
BLOODY RUN (OR BANK) WATERSHED CHARACTERISTIC PARAMETERS
"STORM DATA"
RAINFALL HYETOGRAPH(S) FROM RAINGAGE STATION(S)
HYDROGRAPHS AND POLLUTOGRAPHS FROM MONITORING STATION(S)
INPUT IS PUNCHED ON CARDS AND ENTERED INTO STORM-WATER MANAGEMENT MODEL.
OUTPUT: HYDROGRAPH(S), POLLUTOGRAPHS, ETC. AT SPECIFIED LOCATION(S) IN SYSTEM
COMPARISONS MADE

Note: The term "storm data" is used in this report as a reference to the basic data that must be changed with each storm in the process of supplying input data to the Model.

Figure 7-7. GENERAL TESTING PROCEDURE

TRANSPORT block.) Other modifications can be noted in Table A-1. (Appendix A)

Three listings of data input are given in Appendix B.

All comparison graphs are presented in Appendix A. Tables A-2 and A-3 should be used to locate graphs of interest.

RESULTS FROM TESTING OF STORM WATER MANAGEMENT MODEL

The comparison of the Storm Water Management Model output with actual (measured) field data formed the basic approach to the testing of the model. In addition to storm hyetographs, other input data variations were involved. Input variations were used in testing the results for the entire Bloody Run Watershed;similar tests were then conducted using a sub-section of Bloody Run, i.e., the Bank Watershed referred to previously.

Table A-1 provides a listing of the comparisons of computer output and actual field data. Each comparison is associated with its appropriate input data variation.

Tables A-1 and A-2 provide a summary and index for graphs comparing the Storm Water Management Model results with field data. In most cases, graphs of the SWMM results and the recorded hydrographs have been plotted; however, in some cases the similarity of previous comparisons made graphs unnecessary.

The comparison graphs are given in Appendix A. The graphs include basic comparisons for hydrographs (flow) and pollutographs (BOD and suspended solids) at various points in the watershed and for varied conditions as tabulated in Tables A-1, A-2 and A-3. A general evaluation of the results is given below.

Sewer Infiltration

Since the program includes an input provision for infiltration into sewers some testing of the sensitivity of this factor was carried out. For example, for the Bank Watershed, the common dry weather infiltration into the sewers from ground water was estimated to be 100 gpm based on dry weather flow information. This was then increased to 2550 gpm for test purposes using the August 4, 1971 storm. Only slight variations in the hydrograph resulted, however. It was concluded that this infiltration factor in many cases may not be significant where flows are large as compared with the infiltration from ground water through sewer joints, etc.

The sewer infiltration factor did, however, have a marked effect on the pollutographs. The concentrations of BOD and suspended solids were significantly lowered with the higher infiltration due to the increased dilution.

Rainfall Infiltration (at Ground Surfaces)

As provided in the SWMM USER's MANUAL, when infiltration data are not available, a standard infiltration-capacity curve should be used. Therefore, in the initial testing, the curve for a pervious residential area (as obtained from ASCE, Manual of Engineering Practice No. 37, "Design and Construction of Sanitary and Storm Sewers," Water Pollution Control Federation, Manual of Practice No. 9, 1969) was used. This curve defines the maximum infiltration rate as 3.00 in/hr, the minimum infiltration rate as 0.52 in/hr, and the decay rate of infiltration as 0.00115 sec^{-1}. These infiltration data were then changed on the basis of infiltration tests actually run in the Bloody Run Sewer Watershed (please refer to section 4 in this report,Determination of Infiltration Capacity Curves for Use in Mathematical Models). Results from this work showed a maximum infiltration rate of 4.75 in/hr, a minimum infiltration rate of 0.445 in/hr, and a decay rate of infiltration of of 0.00281 sec^{-1} for pervious areas. A comparison was made using these revised infiltration data for the May 7, 1971 storm at both the Bank and Bloody Run Watersheds. The results from these new input data showed little difference in hydrographs as compared to tests done using the original standard infiltration-capacity curve.

Imperviousness and Surface Detention Depths

Surface detention depths are taken by the program as 1/16-inch for impervious areas and 1/4-inch for pervious areas, unless specified at other values by the user. This was done for both watersheds by changing the variable PCTZER, the per cent of impervious area with zero detention depth. These variations were found to have considerable effect on the hydrographs. For example, when the Bank Watershed value of 40% (estimate of actual value) was changed to 1% using the August 4, 1971 storm as input, it was found that the hydrograph peak dropped from 12.0 cfs to 0.6 cfs. Little effect was noted with regard to changes in quality, however.

EVALUATION AND DISCUSSION OF THE EPA SWMM

As a general basis for the evaluation of errors between the Model and the actual field data, Table 7-6 has been developed which provides an error analysis of the basic comparisons. For example, for hydrographs (flow vs time) the percentages of error were calculated for deviations in the time of the peak, the amplitude of the peak and the total volume under the hydrograph. In each case, the field hydrograph data were assumed to be correct and were used as the bases for computing the error. The following observations are made with regard to this analysis.

Average Errors--Bloody Run Watershed Testing

For the total Bloody Run Watershed testing at the various stations, the computed magnitudes of hydrograph peaks varied from 73% below to 490% above the field data. The average error for the comparisons was +86%.

The timing of the hydrograph peaks was fairly good with an average error of +1.5%. The time error was computed by dividing the absolute deviation of the peaks by the time interval of the hydrograph base. The average absolute error was 30 minutes. For the total volume under the hydrographs, the average error was estimated at +50%.

The water quality data exhibited better agreement. In the case of suspended solids, the pollutographs agreed reasonably well with the field data in the magnitudes of the peaks; the average error was calculated to be -11%. The timing of the peaks show an average error of 6.1%.

Predicted BOD concentrations ranged from 90% below to 30% above the field data with an average error of -23%. The timing of the BOD pollutograph peaks was good as indicated by an average error of -2.8%.

The addition of gutter/pipe data did not have any significant effect on the results. As a matter of expediency, these smaller pipes initially were not included in the input data and then were added to test the sensitivity of the Model.

One possible source of error in the Bloody Run total watershed results could be in the representability of the rainfall data over the entire watershed for a particular storm. For distribution of the rainfall input data over the watershed the Thiessen method was used to associate each of the three raingage hyetographs with its proper subcatchment. This representation is considered to be a valid approach. It is, however, likely that the storm intensity varied over the watershed in many cases, and this could account for some discrepancies between the Model results and the field data in the graphs.

Naturally, as the area becomes smaller, this factor with

TABLE 7-6
ERROR ANALYSIS
STORM WATER MANAGEMENT MODEL vs FIELD DATA

General Notes: (-) indicates computed value was "low" or "early".
(+) indicates computed value was "high" or "late".

Method of Computation:

% Time Error = Difference in time between major computed and measured peaks divided by the time base of the hydrograph or pollutograph.

% Amplitude Error = Difference in flow rate or concentration between major computed and measured peaks divided by the amplitude of the measured (field) peak.

% Volume Error = Difference in area under computed and measured hydrographs divided by the area under the measured hydrograph.

(*) indicates error not calculated due to sporadic field data or multiplicity of peaks.

Notes on Bloody Run Testing: All errors refer to comparisons where the computer runs did not include gutter/pipes. There was no appreciable improvement when these smaller pipes were added.

Notes on Bank Testing: The Bank Watershed results exhibit smaller errors because of greater discretization and better representation of rainfall. For Bank hydrograph errors comparisons were made between measured and computed data at the Bank 54" station without gutter/pipes. The addition of gutter/pipes had little effect on the output (computed) hydrographs.

TABLE 7-6 (continued)

Bloody Run Hydrographs

Storm/Station	Error in Time of Major Peak	Error in Amplitude of Major Peak	Error in Hydrograph Volume
Storm of 10/20/70			
OUTLET	+10%	-10%	-10%
Storm of 11/9/70			
OUTLET	+11%	-30%	-5%
12' x 9'	0%	+150%	+159%
11' x 9'	+15%	+206%	+117%
BANK 78"	+10%	+78%	+43%
Storm of 11/14/70			
OUTLET	0%	-50%	-16%
12' x 9'	-18%	+10%	-10%
11' x 9'	+1%	+122%	+158%
BANK 78"	-10%	+250%	+309%
Storm of 5/7/71			
OUTLET	-10%	-50%	-80%
12' x 9'	-1%	+89%	+42%
11' x 9'	+3%	+52%	+12%
BANK 78"	+8%	+12%	-17%
Storm of 5/13/71			
OUTLET	+5%	-38%	-46%
12' x 9'	+5%	+134%	+67%
11' x 9'	+6%	+182%	*
BANK 78"	+2%	+212%	+158%
BANK 54"	+4%	-25%	-28%
Storm of 8/25-26/71			
OUTLET	-4%	+67%	*
11' x 9'	-4%	-122%	-22%
BANK 78"	+10%	+67%	*
BANK 54"	-2%	+490%	*

Bloody Run Pollutographs--Suspended Solids

Storm/Station	Error in Time of Major Peak	Error in Amplitude of Major Peak
Storm of 10/20/70		
OUTLET	+40%	-42%
BANK 54"	0%	-3%

TABLE 7-6 (continued)

Storm of 11/9/70		
BANK 78"	-8%	-6%
BANK 54"	+2%	+26%
Storm of 11/14/70		
BANK 78"	-40%	-23%
BANK 54"	-33%	+14%
Storm of 5/7/71		
OUTLET	+10%	-21%
12' x 9'	+2%	-54%
Storm of 5/13/71		
OUTLET	+12%	-15%
Storm of 8/25-26/71		
OUTLET	-2%	+400%
BANK 54"	+1%	+1740%

Bloody Run Pollutographs--BOD

Storm/Station	Error in Time of Major Peak	Error in Amplitude of Major Peak
Storm of 10/20/70		
OUTLET	0%	-62%
BANK 54"	*	-92%
Storm of 11/9/70		
12' x 9'	-1%	-42%
11' x 9'	-21%	+228%
BANK 78"	+1%	-51%
BANK 54"	+1%	+9%
Storm of 11/14/70		
12' x 9'	*	-93%
11' x 9'	*	-95%
BANK 78"	*	-90%
BANK 54"	+19%	-85%
Storm of 5/7/71		
OUTLET	*	-77%
12' x 9'	*	*

TABLE 7-6 (continued)

Storm of 5/13/71		
OUTLET	-16%	-50%
Storm of 8/25-26/71		
OUTLET	*	+30%
BANK 54"	*	*

Bank Hydrographs (BANK 54" Station)

Storm	Error in Time of Major Peak	Error in Amplitude of Major Peak	Error in Hydrograph Volume
Storm of 5/7/71	+8%	+47%	+22%
Storm of 5/13/71	+4%	-25%	-7%
Storm of 8/3-4/71	-6%	-21%	-29%
Storm of 12/6/71	+15%	-33%	+17%

Bank Pollutographs (BANK 54" Station)--BOD

Storm	Error in Time of Major Peak	Error in Amplitude of Major Peak
Storm of 8/3-4/71	+28%	+7%

regard to distribution of the rainfall intensity becomes minimized. This seems to be borne out by the fact that better correlation between the Model and field data was obtained in working with the 105-acre "Bank" Watershed, where storm data were collected by one gage. The gage concentration for the smaller area was then 105 acres/gage while the concentration for the Bloody Run Watershed was 2380 acres/ 3 gages or 793 acres/gage.

Average Errors--Bank Watershed Testing

The 105-acre Bank Watershed was tested in order to examine results for an area substantially smaller than the total watershed. One raingage in the near vicinity was used for the storm hyetographs for this area while three gages were used for the total watershed.

In general, the Model's predictions for this smaller drainage area were much closer to the field observations. For the hydrographs, the volume error was only + 0.75%; the error in timing of the peaks was +5.4%; and the amplitude (height) was -8% on the average. This seems to be reasonably good and indicates that better results may be expected with smaller areas.

The pollutograph results also seemed to be quite good as compared to the field data, particularly when the gutter/ pipes were included in the pipe system input.

Part of the improvement in the correlation of results may be accounted for in the greater discretization of the pipe network. In addition, it must be recognized that the rainfall data for the Bank Watershed were more representative of the actual storm events. As indicated above, a gage concentration of 105 acres/gage is much better than 793 acres/ gage (the Bloody Run gage concentration).

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the resulting correlations and preceding error analysis, a number of conclusions with regard to the Model are listed below.

1. When a large area is being considered, such as the 2380-acre watershed, the discretization of the pipe network into great detail (such as the inclusion of gutter/pipes) does not seem to be necessary if the simulation of hydrographs is the main concern.

It has also been observed that variations in the input data for infiltration rates, infiltration constants, and dry weather flow do not significantly affect hydrographs for large areas, i.e., 2380 acres. However, these factors, along with pipe network discretization, do have an effect on the prediction of pollutographs.

2. When small areas are considered, such as the 105-acre Bank watershed, the discretization of the pipe network (inclusion of the gutter/pipes) becomes significant and increases both the hydrographs and pollutographs. However, the pollutographs seem to be more sensitive to this input than the hydrographs.

Changes in rainfall infiltration and sewer infiltration have a greater effect on both hydrographs and pollutographs in the case of the smaller area. The effects of these changes are quite significant in connection with the pollutographs.

3. The overall comparison of results indicates that a greater degree of accuracy can be expected in using the Model for smaller areas such as up to several hundred acres. As the size of the watershed increases, larger departures in accuracy seem to result for both hydrographs and pollutographs.

4. Although the comprehensive nature of the Model is a desirable feature for the more sophisticated users, those less familiar with work of this nature will find it rather complex.

5. Data analyses and interpretation are essential.

6. Quality and perhaps quantity variations due to geographical or seasonal differences must be properly characterized.

7. The real time occurrence of the storm event (hour of the day) significantly affects the quality of combined sewer overflows.

8. Idealized "design storms", i.e., the traditional 2-year, 5-year, and 10-year storm, may give a poor representation of the quality characteristics of overflows by failing to represent the time of occurrence and shape of the hydrographs.

9. The first flush effect is a function of the antecedent dry weather flow, the transport system geometry, and the design event. The dry weather flow provides the source, the transport system determines location and rate of solids accumulation, and the design event determines the rate of uptake and travel time to the overflow point.

10. Quality concentrations alone are not adequate indicators

of the pollution potential of combined sewer overflows and storm water discharges because of the high variation in flow rates during design events. Mass (flow times concentration) discharge rates are generally more significant for the long-term effects, and concentrations are perhaps more significant for the short-term shock effects.

11. The areal distribution of rainfall can significantly influence runoff quality.

12. The rainfall data used for computations with the Model are basic to the results which may be expected. It is important that the hyetograph be representative of the location and the desired storm frequency. Because of the importance of the rainfall input, Section 3 has been included in this report entitled "Development of Design Storm Hyetographs for Use in Mathematical Models".

In most urban watershed areas, it is unusual to have extensive hyetograph data from an automatic rainfall gage within the watershed. Therefore, it is necessary to select design storms which are representative of the local climatic conditions. As pointed out previously in this report, as the watershed area becomes larger, there is less chance that the storm will equally cover the entire area with the same pattern and this factor may affect the expected results.

13. It is to be recognized that rainfall infiltration becomes a more and more significant parameter as the percentage of pervious area increases. Therefore, this parameter may be a source of error if not properly characterized. Because of this possibility, infiltration capacity tests were made in a pervious area of the Bloody Run Watershed and these results have been included in this report as an aid in determining this input parameter. This information is included in Section 4 entitled "Determination of Infiltration Capacity Curves for Use in Mathematical Models".

SUGGESTED MODIFICATIONS OF THE EPA STORM WATER MANAGEMENT MODEL

Suggested modifications of the EPA SWMM are described below under two headings: The first concerns a basic alteration of the computer program itself, the second deals with possible improvements in the SWMM USER's MANUAL, Vol. III

1. Suggested Modifications of the Main Program

a. With respect to the quantity aspects of hydrologic runoff, the model reacts significantly only to the runoff-affecting factors: rainfall, slopes, pipe network, etc. Information related to flow quality is negligible in its effect on flow quantity (discharge) output. This is to be expected. Yet this flow quality data, which includes catch-basin pollutant concentrations, street cleaning information, land use data, gutter lengths for each subarea, and much more, must be given as input if the program is to produce any significant results. Skipping some of these data may cause the job to be terminated, and will certainly produce error messages.

b. Ideally, it should be possible for the user (when and if he desires) to delete the above-mentioned quality data which are known to have little or no effect on the output hydrographs (flow quantity vs time). (The surface quality data required by the RUNOFF block of the program is an example). This would save time and effort when preparing the data for limited output, i.e., flow quantity information only. Computer time would be greatly reduced as well if the program could omit the extensive quality-related computations.

c. Relatively minor modifications of the program would make this simplification possible. As the Model now stands, the user will come across several situations in the USER's MANUAL (Vol. III of the Storm Water Management Model Series)where the data required for certain card groups are optional, depending on previously supplied information. Two examples of this in the RUNOFF block of the program are card groups 10 and 12. Group 10 may be skipped if the variable "NSAVE" in group 9 is zero. Group 12 may be skipped if "NPRNT" from group 11 is zero. The modifications suggested here would enable a programmer to skip, in a similar manner, large sets of data such as card groups 15 through 18 (Surface Quality data). A previous card group would set a variable at 0 or 1 depending on whether the run would be quantitative only, i.e., hydrographs without regard to pollutants, or inclusive of flow quality aspects also for which the program was originally designed. The computer, after reading the variable (0 or 1) would eventually encounter an IF statement which, if the number was 0, would cause the computer to skip a large portion of unnecessary computations (viz. quality-related information) and proceed to the next flow quantity (hydrograph) operation. Hence, by supplying the proper READ statement and as many IF/GO TO statements as are necessary, the program could be adapted to make purely quantitative uses an important and useful option.

d. Based on testing done at the University of Cincinnati, modifications of the program itself seem unnecessary for qualitative use. That is, pollutographs are definitely sensitive to the required quality data, and predicted pollutographs were reasonably close to their measured counterparts. Since the flow quality results are dependent on the quantity of hydrologic runoff, there is no question of optionally deleting any of the flow quantity data. If the user is interested in flow quality (pollutographs) he must use the program in its full capacity as intended by its designers.

2. Commentary on the Storm Water Management Model User's Manual

a. The User's Manual assumes a considerable amount of expertise on the part of the Model user. He must be well versed in computer operations as well as knowledgeable in the areas of hydraulics, sanitary engineering, and hydrology.

This problem may be remedied by including a comprehensive glossary defining the technical terms used in the manual. This will do away with the necessity to refer to the other three volumes of the Model in order to clarify terminology questions.

b. The User's Manual presently includes examples of test runs organized according to the program "blocks". That is, there is sample input and output for all five program blocks, but these examples are separate and do not really help when the user is concerned with his own non-simplified problems, where three or more blocks are being used at a time.

Examples illustrating the use of the program blocks RUNOFF and TRANSPORT in combination would be helpful, especially if these examples included test runs with and without gutter/pipe information (see p. 60, User's Manual). Examples of other possible combinations including the STORAGE and RECEIVING WATER blocks would also be useful. A special appendix could be added to the User's Manual giving these more complex examples.

c. In summation, the User's Manual would be more useful if written in such a way that it could stand alone. The data preparation would proceed in a much smoother manner if this were the case.

AIDS IN DATA PREPARATION FOR USERS OF SWMM

The following information is intended to clarify some of the problems confronting the user who may be unfamiliar with the Storm Water Management Model and its instructions for data preparation. The Model user will no doubt have questions concerning the preparation of the data cards and the following notes are organized according to program "Blocks" and "Card Groups" as in the User's Manual, Vol. III, Storm Water Management Model, September, 1971. Part (a) deals specifically with terminology and related problems encountered in the preparation of data cards according to the instructions in the User's Manual. Part (b) is concerned chiefly with the clarification of problems not specifically encountered in part (a).Finally, in part (c) a list of data sources is provided, if and when the user finds that his previously compiled information is inadequate.

a. Terminology

EXECUTIVE BLOCK (p. 23 ff.)

Card Group 2: These variables do not enter into calculations. Average daily Dry Weather Flow (DWF) was computed on a weighted average basis (p. 23).

Card Group 3: Amount of rainfall=Area under hyetograph (in/hr vs time) (p.23).

RUNOFF BLOCK

Card Group 2: "Integration Period": same as time interval for hyetograph (p. 59).

Card Group 4: Data for hyetograph cards are mid-interval values (p. 59).

Card Group 5: Predominant slope, i.e., slope associated with longest length, seems most logical if more than one slope is given for a subarea and further division is not desired (pp. 59-60).

Card Group 7: Skipping this card group is termed "coarse discretization" by the User's Manual. Data gathering (for "fine discretization") may involve working with the public works department in order to find maps that show the gutter/pipes in question. A blank card must be placed before and after this card group, even if it is not included as input (p. 60).

Card Group 17: A "catchbasin" is a gutter inlet. Obtain data from city maps, public works department, or estimate (p. 62).

TRANSPORT BLOCK (p. 169 ff.)

Card Group 12: See User's Manual, pp. 5-7 (p. 171).

Card Group 13: "Dry Weather Days"--number of days that cumulative precipitation is less than 1.0 inches (p. 171).

Card Group 15: "Barrels"--use default value. This is the number of manholes per pipe actually (always 1.0 the way the problem is set up). The term "barrel" refers to the concrete cylinder for a manhole (pp. 172-173).

b. Definitions

In order to clarify some terminology and data requirements, the following definitions may be found in the User's Manual on the pages indicated:

"Discretization" (p. 49).

Infiltration capacity curves, pervious areas only (p. 54).

"The number of catchbasins (gutter inlets) per acre may be estimated from visual observation or obtained from a public works department..." (p. 57).

"The length of gutters within each subarea may be obtained by scaling them from a street map." (p. 57).

"The quality constituents considered for simulation are the 5-day BOD, total suspended solids, total coliforms and DO...Other parameters such as COD, volatile suspended solids, settleable solids, and fecal coliforms, can be developed by paralleling the structures of their modeled counterparts." (pp. 5-7).

"At the point where the water quality constituents are introduced and are to be routed, the transport block must be used, since the runoff block is not able to route contaminants through a pipe network." (p. 52).

Gutter/pipes - "Pipes smaller than 2-3 ft with no back water effects, flow dividers, or lift stations, are usually designated as gutter/pipes for computation by the Runoff Block. These pipes are not connected to one another by manholes, but join directly and lead to an inlet manhole for further routing by Transport." (p.55).

c. Data Sources

1) Previously compiled information. Know what has been done and where to find it.

2) Lab personnel (Sanitary Engineers) may be able to supply the needed data relating to pollutant discharges, and concentrations.

3) The User's Manual should be read thoroughly. It gives suggestions as to how to go about estimating various parameters when they are unavailable.

4) Maps can be obtained from the City Sewer Dept., or Dept. of Public Works which contain all the information conceivably needed to describe a particular pipe network.

5) When questions arise as to how critical certain data are, it is best to check the Program Listing, Volume IV, Storm Water Management Model, October, 1971. This volume contains all the equations used by the Model.

SECTION 8

TESTING AND COMPARISON OF VARIOUS METHODS FOR THE DETERMINATION OF URBAN STORMWATER RUNOFF

INTRODUCTION

Design of urban storm drainage systems for cities has been carried out for many years on the basis of empirical relations and experience. Public concern and the need for stormwater pollution control have stimulated much research interest in the area of urban hydrology in recent years.

The engineering literature contains a number of analytical methods designed to yield the hydrograph of urban stormwater runoff, given the input rainfall hyetograph and the drainage basin structure. These methods range from the very simple Rational Formula to the very detailed EPA Storm Water Management Model. To a varying degree most of these methods rely upon empirical relationships and experience. Very few methods utilize the basic equations characterizing the hydrologic cycle.

A brief background on the various methods developed for the design and analysis of urban storm drainage systems as they evolved through this century is presented herein and the most reliable and up-to-date methods are compared in three case studies.

BACKGROUND OF VARIOUS METHODS

Empirical Methods. Late in the 19th century, engineers developed empirical formulas to determine design discharges for storm sewers. One of them, the Rational Formula, is perhaps the most widely used even today. The Rational Formula is expressed as $Q = CiA$ where Q is the peak discharge in cfs, C is the runoff coefficient depending on the characteristics of the drainage area, i is the uniform rate of rainfall intensity in inches per hour for a duration equal to the time of concentration, and A is the drainage area in acres. The time of concentration is the time required for the surface runoff from the remotest part of the drainage area to reach the point under design. The runoff coefficient represents the multiplier of a 100% runoff peak (with no infiltration or storage) required to obtain the design peak. The personal judgment required for the estimation of the proper value of C is one of the major drawbacks of the rational method together with the evaluation of the time of concentration. The rational method determines the peak of the discharge hydrograph but not its shape, and the larger the drainage area the more inaccurate the result will be.

In 1932, Gregory and Arnold [5] developed a modification of the rational method by taking into consideration the shape and slope of the catchments, the pattern of the drainage system, and the elements of channel flow.

Los Angeles Hydrograph Method. In 1944, Hicks [7] reported a method of computing urban runoff for the Los Angeles area. The method, called the Los Angeles Hydrograph Method, is based on experimental work for the determination of the infiltration capacities for various local soils and for the determination of overland flow over various types of soils at different slopes. For determining antecedent rainfall conditions, a weighted moisture factor was used related to the 60-day antecedent seasonal evaporation and the weighted 60 day antecedent precipitation. A measure of depression storage was also determined from some of the large area rainfall-runoff records and was incorporated in the design procedure. The time of concentration was taken as the sum of the time of lot runoff, the time of flow in gutters or channels to an inlet, and the time of flow in the storm sewer. For the analysis of experimental data, charts were provided for the derivation of the time of concentration. Since this method is based on data and information collected for the mountainous and alluvial plain areas in the vicinity of Los Angeles, it is not readily applicable to other drainage areas.

Izzard's Method. In 1946, Izzard [9] developed a method to simulate overland flow across sloping plane surfaces at unsteady state. The equations derived are based on experimental data limited to slopes less than 0.04 and to situations where $iL<500$ where i is the rainfall intensity in inches per hour and L is the length of overland flow in feet. The results from these experiments were found to agree closely with those obtained by the Los Angeles Hydrograph Method. Izzard developed both the rising and recession portions of a dimensionless overland flow hydrograph which, because of its simplicity, is a rather useful design tool.

Chicago Hydrograph Method. In 1960, extensive studies of a hydrograph method were made by the city of Chicago. A detailed description of the so-called Chicago Hydrograph Method was presented by Tholin and Keifer [18]. This Method simulates flow on grassed areas, roofs, paved streets, and through gutters and storm drains to the basin outlet. The method considers that the time of beginning of excess rainfall occurs when the antecedent mass of rainfall equals the antecedent mass of infiltration computed by Horton's equation. A normal distribution was used to determine the probable depth distribution relationship of depressions of a given mean depth at any random location. The overland flow supply mass curve is obtained after subtracting the infiltration and depression storage. Izzard's relationships were used in a routing-storage procedure to obtain the hydrograph of overland flow from the supply mass curves. The gutter-storage volume is determined as a function of inflow and outflow. This function and the continuity equation are used for the gutter routing. A time-offset method is used for the routing through the lateral sewers. Each inflow hydrograph is shifted the amount of time required for water to travel from its point of entry into the sewer to the point under consideration. The estimation of this routing time which was assumed to be eight minutes is a weak point of the method. The same routing method is used for the main sewer. Realizing that only a few of the 13 parameters involved in this method affected the peak rate, a series of charts was prepared to facilitate the use of the Chicago Hydrograph Method.

Inlet Method. In 1963, Kaltenbach [11] summarized the Inlet Method studied at the Johns Hopkins University. The method determines the peak runoff rate q to each inlet by using the rational formula. The inlet hydrographs are assumed to be triangular with height q and base $2T$ where T is the time from the beginning of intense rainfall to the end of the maximum rainfall intensity. The routed inlet hydrographs at the design point have also triangular shape but with a reduced peak and a lengthened base on the rising limb by an amount equal to the time of wave travel in the sewer assumed to be $0.8\ L/V$ where L is the sewer length and V is the mean velocity of flow in the sewer. The total hydrograph at the design point is obtained by adding the routed inlet hydrographs assuming a common time origin. The method yields better results when applied in impervious areas. Some of the limitations of this method are [10]:inlet areas less than three acres; drainage areas less than one square mile; imperviousness 30 to 60 per cent; gutter slopes of two to seven per cent; steep roofs and surface slopes so as to avoid significant surface storage.

Unit Hydrograph Method. In 1932, Sherman [16] published the Unit Hydrograph Method. A unit hydrograph is a hydrograph of surface runoff resulting from a unit storm, i.e., a short and intense storm of such duration that the period of surface runoff is not appreciably less for any rain of shorter duration. The duration of a unit storm must not exceed the period of rise of the hydrograph [21]. For an urban basin a unit hydrograph can be developed if observations are made of actual rainfall and runoff. The method makes use of distribution graphs which have the same time scale as a unit hydrograph and ordinates, which are the percent of the total surface runoff that occurred during successive, arbitrarily chosen, uniform time increments. All unit storms, regardless of their magnitude, produce nearly the same distribution graphs for a drainage area. These graphs serve to convert any expected volume of surface runoff into a hydrograph of river discharge. In 1962, Eagleson [4] developed unit hydrographs for sewered areas and discussed the degree to which their characteristics can be related to properties of the urban basin.

RRL Method. The Road Research Laboratory Method [17] considers only the impervious areas of the basin that are directly connected to the storm drain system. The rest of the basin, such as lawns, parks or impervious areas draining onto grass, are disregarded. For the directly connected impervious areas, flow velocities are computed for plane surfaces, gutters, and lateral and main storm sewers. These velocities are used to construct on a map lines of equal travel time to the outlet of the basin which are called isochrones. The areas between the isochrones are measured and cumulated to provide a time-area diagram which together with the applied rainfall pattern is used to construct a runoff hydrograph called the virtual runoff hydrograph. Then a discharge-storage relation is used to account for the effects of storage on the runoff within the basin. Finally, a simplified one-step storage-routing procedure is used to allow for the effects of temporary storage within the sewer system.

UCUR Model. With the computer evolution, in 1970, Papadakis and Preul [13], [14], [15], developed a mathematical model termed herein as the University of Cincinnti Urban Runoff(UCUR)Model which is able to simulate specific hydrologic conditions using a deterministic method with a parametric system synthesis approach. Input parameters include the rainfall hyetograph and physical data describing the watershed and the drainage system. The UCUR Model consists of five sub-models which simulate individually the processes involved, i.e., infiltration, surface retention, overland flow, gutter flow and routing through the sewer system. The infiltration sub-model uses Horton's equation to calculate the infiltration capacity curve. The time of the start of rainfall excess is calculated by comparing the amount of water precipitated with that infiltrated. For the surface retention submodel, an equation was developed relating depression storage supply with time. The development of the overland flow submodel is based on the conservation of mass and the continuity equation. An equation by Crawford [2] was also employed relating the overland flow depth with the surface detention. For the gutter flow submodel, the continuity equation was used assuming steady flow conditions. This assumption is believed to be justified since the change in gutter flow depth with respect to time is relatively slow under most runoff conditions. In the sewer-routing submodel, routing through the laterals and main sewer is accomplished by shifting the inflow hydrographs in time from one inlet to the next downstream [13]. Each preceding hydrograph is then added to the next junction downstream until the full system is encompassed at the final outlet. This procedure results in slightly higher peak flows than those calculated by using more exact methods. However, the simplicity of the procedure is advantageous. The UCUR Model was tested initially on small watersheds [14] and later on urban watersheds of several thousand acres in area using the computer IBM 360/65 of the University of Cincinnati.

EPA Storm Water Management Model. In 1971, under the sponsorship of the Environmental Protection Agency (EPA) Water Quality Office, a consortium of contractors developed a comprehensive mathematical model termed EPA Storm Water Management Model(EPA SWMM)[12]. The model is capable of simulating urban stormwater runoff and combined sewage overflow phenomena by using a high-speed digital computer. Rainfall hyetographs and system characteristics are used as input to the model, which calculates runoff quantity and quality. In the following only quantity aspects of the EPA SWMM are considered.

The programming arrangement consists of an executive or control block and four computational blocks: Runoff, Transport, Storage, and Receiving Water. The Runoff Block computes the stormwater runoff and its characteristics for a given storm for each subcatchment and stores the results in the form of hydrographs at inlets to the main sewer system. These computations are based on any rainfall hyetograph, antecedent condition, land use and topography. The Transport Block sets up pre-storm conditions by computing dry weather flow and infiltration and distributing them throughout the conveyance system. The block then performs its primary function of flow routing, picking up the runoff results, and producing combined flow hydrographs for the total drainage basin and at selected points within the system. The computations of this block rely on Manning's equation and the assumption of continuity. The Storage Block uses the output of the Transport Block and modifies the flow at a given point or points according to the pre-defined facilities provided. Costs associated with the construction and operation of these facilities are computed. The Receiving Water Block accepts the output of the Transport Block, or the modified output of the Storage Block, and computes the dispersion and effects of the discharge in the receiving river, lake or bay.

The Storm Water Management Model was initially tested by Metcalf & Eddy, Inc., et al [12] and verification research with urban watersheds continued at the University of Cincinnati using an IBM 360/65 digital computer.

SIMULATION TECHNIQUE DIFFERENCES BETWEEN UCURM AND EPA SWMM

In many respects the University of Cincinnati Urban Runoff Model and the EPA Storm Water Management Model are similar in structure. However, there are differences in the simulation of the individual hydrologic processes involved.

The EPA SWMM simulates the infiltration process by using Horton's equation. No time-offset is used to adjust Horton's infiltration equation if the initial rainfall intensity is less than the initial infiltration capacity. The infiltration potential is satisfied by the rainfall and by water temporarily stored on the ground surface. Thus, infiltration is allowed to continue after rainfall has ceased. The UCUR Model also uses Horton's equation. However, a time-offset method is implemented to adjust Horton's equation when necessary. The time-offsetting influences favorably the initial part of the rising limb of the simulated outflow hydrograph. Infiltration is satisfied only by rainfall and no allowance is made to satisfy infiltration by surface detention which may result to some overestimation of the runoff.

The EPA SWMM assumes that no overland flow occurs before all depressions are filled with rain water. However, a percentage of the impervious areas is considered as having zero depression storage capacity in order to artificially simulate immediate runoff. Water in depression storage on pervious areas can be depleted by infiltration. The UCUR Model does not allow depression storage to be depleted by infiltration. However, it employs an exponential relationship which assumes that the rate at which depression storage is filled is proportional to the unfilled volume. This assumption permits overland flow to start before all depressions are filled.

The EPA SWMM assumes uniform depth of overland flow and uses Manning's turbulent flow equation and continuity to perform a quasi-steady state storage routing. The UCUR Model considers overland flow profiles with increasing depth along the length of an overland flow plane. Manning's turbulent flow equation, continuity and an empirical relation are used to simulate the process.

The EPA SWMM considers that each gutter is fed uniformly along its length and uses Manning's turbulent flow equation and continuity to perform the storage routing. The UCUR Model assumes that the outflow is equal to the sum of the inflows which is a good approximation for gutter grades other than very flat.

The EPA SWMM uses Manning's equation based on the slope of the energy grade line and continuity to simulate the flow through the sewer pipes. If flow in a pipeline is supercritical the outflow is simply set equal to the inflow. The UCUR Model does not attempt a storage routing but computes the weighted average velocity to find a lag time for the inflow hydrographs. This technique is good enough except in cases where backwater effects are predominant.

These are in brief the major differences in the simulation techniques employed by the EPA SWMM and the UCURM.

COMPARISON OF VARIOUS METHODS IN THREE CASE STUDIES

Chicago, 10-Acre Tract. A 10-acre drainage area was selected to compare several of the most reliable methods of determining urban stormwater runoff quantity because detailed data were available and because it is typical of the Chicago street pattern and may be typical of some other cities. The catchment has been divided into six basic types of subcatchments:

Type	Description	Length (ft)*	Slope*	Manning's n
I	Dense blue-grass turf, pervious	16	0.0100	0.350
II	Dense blue-grass turf, pervious	40	0.0100	0.350
III	Dense blue-grass turf, pervious	70	0.0100	0.350
IV	Street pavement, impervious	17	0.0200	0.013
V	Alley pavement, impervious	8	0.0375	0.013
VI	Common composition roofing, impervious	12	0.6670	0.012

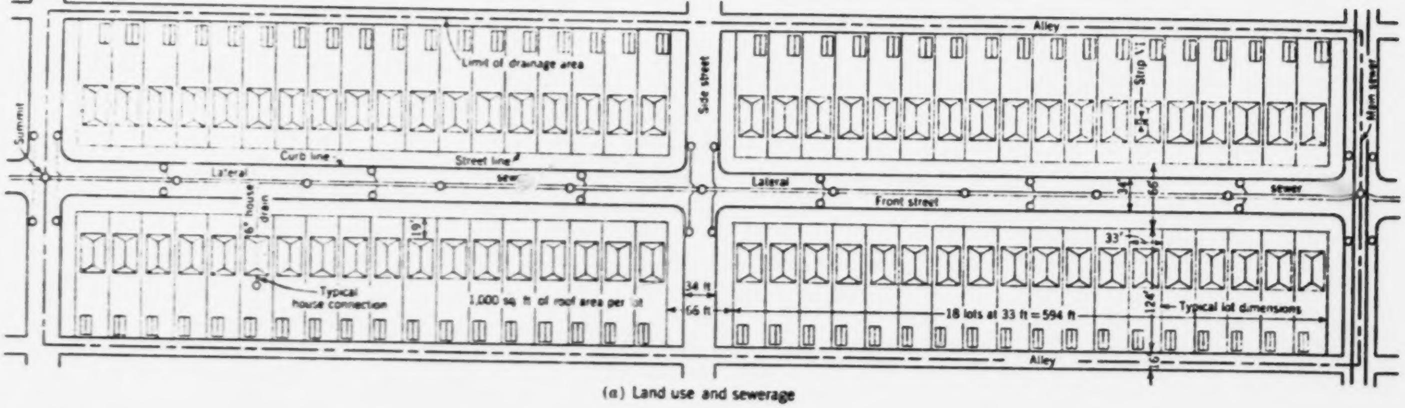
(*Parallel to the direction of overland flow)

The 10-acre catchment constitutes 5.37 acres (53.7%) of pervious areas and 4.63 acres (46.3%) of impervious areas. Building roofs are not discharging through the downspouts on the lawn areas, but are connected directly to the underground sewers. To reduce the amount of computations, six roofs were considered directly connected with each catch basin. The garage roofs, because they discharge onto backyard lawns, were considered a part of the pervious areas. The lateral sewer is of circular cross section with a diameter of three feet, a slope of 0.0045 ft/ft, and a Manning's roughness coefficient of 0.013. Land use, geometry and directions of flow of the drainage area are shown in Figure 8-1, taken directly from the ASCE M & R No. 37 [3].

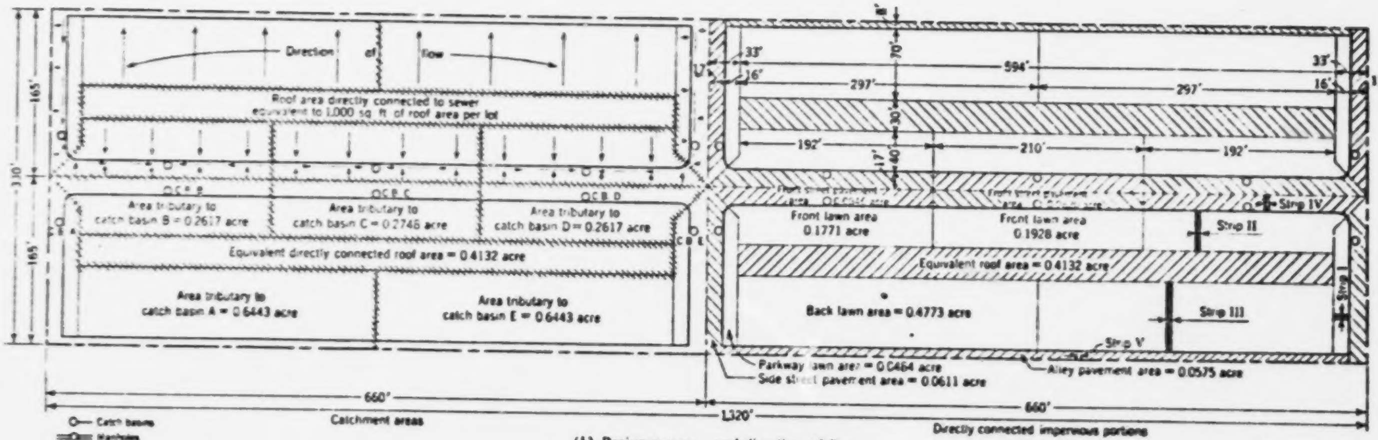
A 181-minute design storm was applied to the drainage basin. For the purpose of comparison, the runoff hydrograph at the outlet of the Chicago 10-acre catchment was calculated by three methods: the University of Cincinnati Urban Runoff (UCUR) Model, the EPA Storm Water Management Model, and the Chicago Method. The resulting hydrographs are plotted in Figure 8-2 together with the 181-minute design storm hyetograph which was applied to the drainage basin. The data cards used as input for the UCUR Model are listed in Table 8-1 (see Appendix for the UCUR Model data cards preparation).

The Chicago Method, the EPA Storm Water Management Model, and the UCUR Model calculated that the peak would occur at about the same time, i.e., 75 minutes from the start of the rainfall as shown in Figure 8-2. The Chicago Method and the EPA Storm Water Management Model calculated a peak of 18.2 and 22.6 cfs respectively. The UCUR Model calculated a peak of 23.0 cfs, which agreed satisfactorily with the EPA Storm Water Management Model. However, the latter divided the Chicago 10-acre catchment into 80 subcatchments, 40 gutters, and 4 pipes, while the UCUR Model used repeatedly 6 basic types of subcatchments, 40 gutters, and 8 pipes, and the computations were made at 5-minute intervals, thereby considerably reducing the computer time required using the IBM 360/65 computer of the University of Cincinnati.

Comparing the peaks obtained by the three above methods with the Rational Formula Q = CiA, the following is obtained:


Summit
Limit of drainage area
Side street
Alley
Strip VI
Main street
Curb line
Street line
Lateral
sewer
6" house drain
Front street
Typical house connection
1,000 sq. ft. of roof area per lot
34 ft
66 ft
18 lots at 33 ft = 594 ft
124'
Typical lot dimensions
33'
19'
16'

(a) Land use and sewerage


Direction of flow
Roof area directly connected to sewer equivalent to 1,000 sq. ft. of roof area per lot
CB B
CB C
CB D
Area tributary to catch basin B = 0.2617 acre
Area tributary to catch basin C = 0.2748 acre
Area tributary to catch basin D = 0.2617 acre
Equivalent directly connected roof area = 0.4132 acre
Area tributary to catch basin A = 0.6443 acre
Area tributary to catch basin E = 0.6443 acre
165'
330'
594'
297'
192'
210'
70'
30'
40'
17'
33'
16'
Front lawn area 0.1771 acre
Front lawn area 0.1928 acre
Strip I
Strip II
Strip III
Strip IV
Strip V
Equivalent roof area = 0.4132 acre
Back lawn area = 0.4773 acre
Parkway lawn area = 0.0464 acre
Side street pavement area = 0.0611 acre
Alley pavement area = 0.0575 acre
660'
1320'
Catchment areas
Directly connected impervious portions
Catch basins
Manholes

(b) Drainage area and direction of flow

Figure 8-1. TYPICAL CHICAGO TWO-BLOCK, 10-ACRE TRACT DRAINAGE PLAN [3]

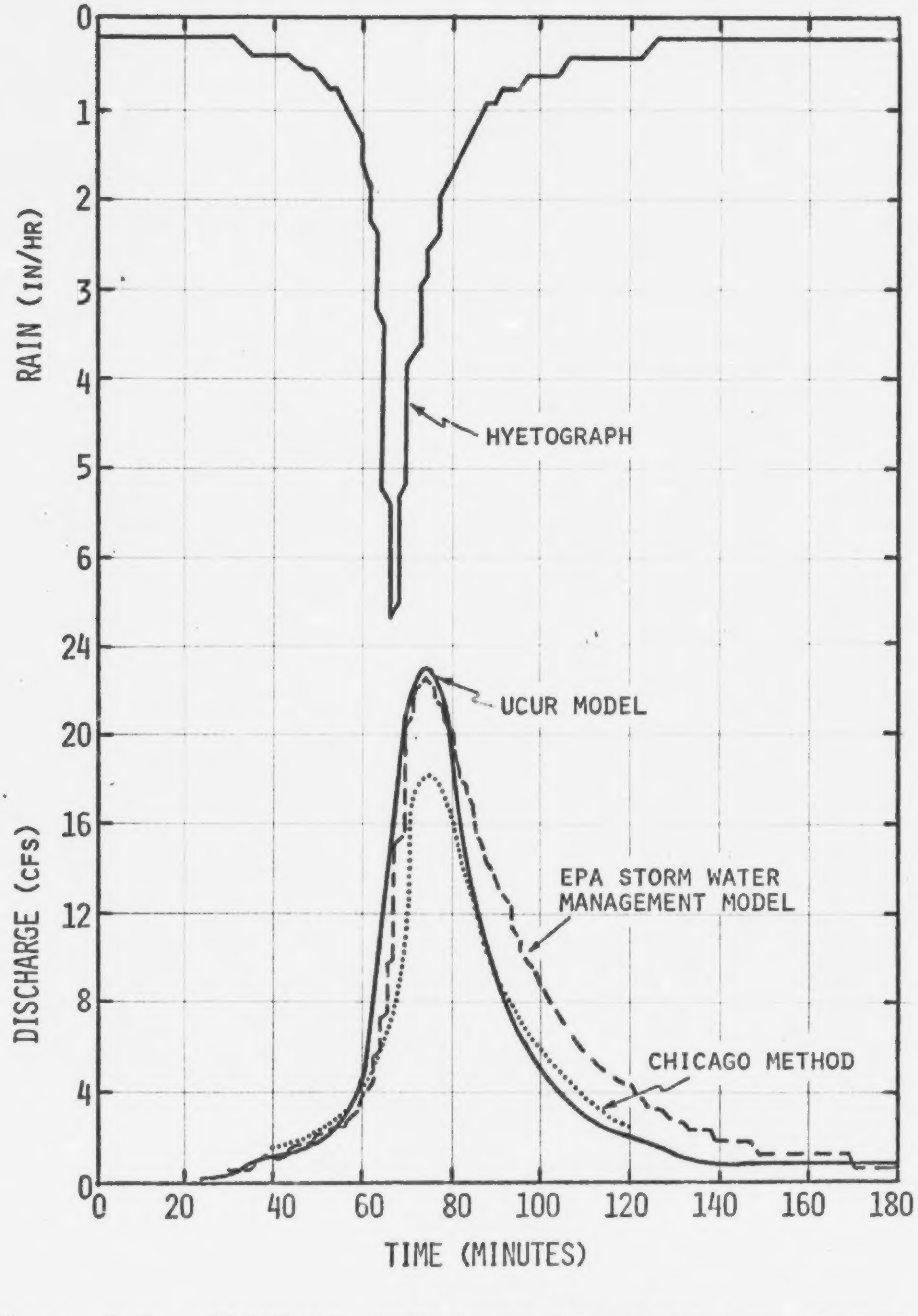


Figure 8-2. RAINFALL HYETOGRAPH AND CALCULATED RUNOFF HYDROGRAPHS. DESIGN STORM. CHICAGO, 10-ACRE TRACT.

Table 8-1. UCUR MODEL DATA CARDS. CHICAGO,10-ACRE TRACT STUDY CASE

```
             CHICAGO              5.0   36
0.20
0.20 0.20 0.20 0.20 0.20 0.20 0.42 0.42 0.50 0.62 0.87 1.63
5.30 5.25 2.80 1.75 1.25 0.88 0.80 0.67 0.50 0.47 0.47 0.47
0.35 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25
     0.53      3.00     0.0697  0.2500  0.0625
30.0    1
   6
1 1   16.0    0.35   0.01
2 1   40.0    0.35   0.01
3 1   70.0    0.35   0.01
4 2   17.0   0.013   0.02
5 2    8.0   0.013 0.0375
6 2   12.0   0.012  0.667
   2  250
   9
   2
   1     2
   3     5      297.0
   1     4      140.0
   3     4
   5     3      297.0
   4     1      140.0
   0     6 30.0
   4
   5     5
   2     4      105.0
   6     6
   4     2      105.0
   7     7
   4     2      105.0
   8     8
   2     4      105.0
  12     6 30.0
   4
   9     9
   2     4      105.0
  10    10
   4     2      105.0
  11    11
   4     2      105.0
  12    12
   2     4      105.0
  12     6 30.0
   4
  13    13
   2     4      105.0
  14    14
   4     2      105.0
  15    15
   4     2      105.0
  16    16
   2     4      105.0
  12     6 30.0
   4
  17    18
   5     3      297.0
   4     1      140.0
  19    20
   3     5      297.0
  1    4      140.0
 21   22
  3    5      297.0
  1    4      140.0
 23   24
  5    3      297.0
  4    1      140.0
  0    6 30.0
  4
 25   25
  2    4      105.0
 26   26
  4    2      105.0
 27   27
  4    2      105.0
 28   28
  2    4      105.0
 12    6 30.0
  4
 29   29
  2    4      105.0
 30   30
  4    2      105.0
 31   31
  4    2      105.0
 32   32
  2    4      105.0
 12    6 30.0
  4
 33   33
  2    4      105.0
 34   34
  4    2      105.0
 35   35
  4    2      105.0
 36   36
  2    4      105.0
 12    6 30.0
  2
 37   38
  5    3      297.0
  4    1      140.0
 39   40
  3    5      297.0
  1    4      140.0
  0    6 30.0
  8
  1    0.0045      0.013  3.0  0.0      120.0    2    1    2    0
  1    0.0045      0.013  3.0  0.0      210.0    2    2    3    0
  1    0.0045      0.013  3.0  0.0      210.0    2    3    4    0
  1    0.0045      0.013  3.0  0.0      120.0    2    4    5    0
  1    0.0045      0.013  3.0  0.0      120.0    2    5    6    0
  1    0.0045      0.013  3.0  0.0      210.0    2    6    7    0
  1    0.0045      0.013  3.0  0.0      210.0    2    7    8    0
  1    0.0045      0.013  3.0  0.0      120.0    2    8    9    0
```

C = coefficient of runoff equal to 0.40 for relatively flat residential areas, 30% impervious [7]

A = catchment area, 10 acres

i = average rainfall intensity in inches per hour for a duration equal to the time of concentration of the catchment

Q = peak flow rate, in cfs

During the 181-minute design storm 2.5 inches of water precipitated on the catchment. This rainfall has a return period of 10 years for the Chicago area [20]. Then the average rainfall intensity for a time of concentration equal to 15 minutes and a return period of 10 years, is i = 5 inches per hour [1]. Therefore, Q = 0.4 x 5 x 10 = 20 cfs, which agrees satisfactorily with the peaks in Figure 8-2.

The three methods gave a comparable rising limb. However, the Chicago Method and the UCUR Model calculated a lower recession than that obtained from the EPA Storm Water Management Model. This can only be accounted for by a larger infiltration loss. Actually the Chicago Method and the UCUR Model compute the mass curve for infiltration loss. Thus, the infiltration capacity tends to be satisfied at any given time even though there is not sufficient rainfall at that moment, under the condition that the accumulated mass precipitated is bigger than the accumulated mass infiltrated. The EPA Storm Water Management Model compares continuously the infiltration with the amount of water existing on the catchment plus the rainfall. If the loss is greater, it is set equal to the amount of water available and the remainder of the computation is skipped.

The EPA Storm Water Management Model maintained the mass continuity within 0.1% of the rainfall. No such information was available on the Chicago Method for comparison. The UCUR Model maintained the mass continuity within an acceptable 2% of the rainfall as has been shown by a series of case studies performed at the University of Cincinnati. Manpower and computer time did not justify an effort for more accuracy.

Oakdale, Chicago. The Oakdale Avenue Drainage Basin in Chicago, Illinois, was selected to test and compare several of the most reliable methods of computing urban runoff quantity because rainfall and runoff have been periodically measured and recorded since 1959 by the Chicago Department of Public Works, Bureau of Engineering [19]. The Oakdale Avenue catchment is an urban drainage area 12.9 acres in size (approximately 2-1/2 blocks long by one block wide) and consists entirely of residential dwellings. The catchment has been divided into six basic types of subcatchments:

Type	Description	Length (ft)*	Slope*	Manning's n
I	Grassed, pervious	35	0.010	0.350
II	Grassed, pervious	35	0.010	0.300
III	Grassed, pervious	50	0.010	0.350
IV	Grassed, pervious	23	0.010	0.350
V	Street pavement, impervious	15	0.020	0.013
VI	Common composition roofing, impervious	11	0.667	0.012

(*Parallel to the direction of overland flow)

The 12.9 acres catchment constitutes 7.05 acres (54.7%) of pervious areas and 5.85 acres (45.3%) of impervious areas. The overall volume of depression storage on the pervious areas was assumed equal to 0.25 inches and on the impervious areas was assumed equal to 0.0625 inches. For normal antecedent conditions of the pervious areas (with no considerable application of precipitation for the preceding three days), the initial infiltration capacity was assumed equal to 3.00 inches per hour; the constant infiltration capacity was assumed equal to 0.53 inches per hour and the decay rate of infiltration was assumed equal to 0.070 per minute. For wet antecedent conditions (considerable rainfall within the preceding 24 hours) of the pervious areas, the initial infiltration capacity was assumed equal to 3.00 inches per hour; the constant infiltration capacity was assumed equal to 0.30 inches per hour and the decay rate of infiltration was assumed equal to 0.089 per minute.

Building and garage roofs in the area drain directly into the underground sewer. The existing 30-inch diameter combined sewer was installed in 1958 and replaced a smaller combined sewer of inadequate stormwater capacity. The slopes of the 30-inch lateral sewer vary from 0.0045 upstream to 0.0030 downstream, and the Manning's roughness coefficient is considered to be 0.013. The plan of the 12.9-acre Oakdale Avenue Drainage Basin is presented in Figure 8-3. In order to use the UCUR Model, the Oakdale Avenue Drainage Basin was divided into 6 basic types of subcatchments, 52 gutters and 9 sewer pipes, and the computations were made at one-minute intervals.

Two storms were applied to the Oakdale Avenue Drainage Basin, the second storm of July 2, 1960, having a duration of 66 minutes, and the storm of July 7, 1964, having a duration of 86 minutes.

For the second storm of July 2, 1960 the runoff hydrograph at the outlet of the Oakdale Avenue Drainage Basin was calculated by three methods: the UCUR Model, the EPA Storm Water Management Model, and the RRL Method. The resulting hydrographs are plotted in Figure 8-4 together with the recorded hydrograph and the 66-minute storm hyetograph. The two peaks calculated by the three methods occurred at about the same time, i.e., around 38 and 62 minutes from the beginning of the rainfall. The recorded first peak, however, occurred some time earlier, 32 minutes from the beginning of the rainfall. The second peak was not recorded. The UCUR, the RRL Method and the EPA Storm Water Management Model calculated a first peak of 18.1 cfs, 14.3 cfs and 15.6 cfs respectively and a second peak of 10.6 cfs, 6.9 cfs and 12.5 cfs respectively. The first peak calculated by the

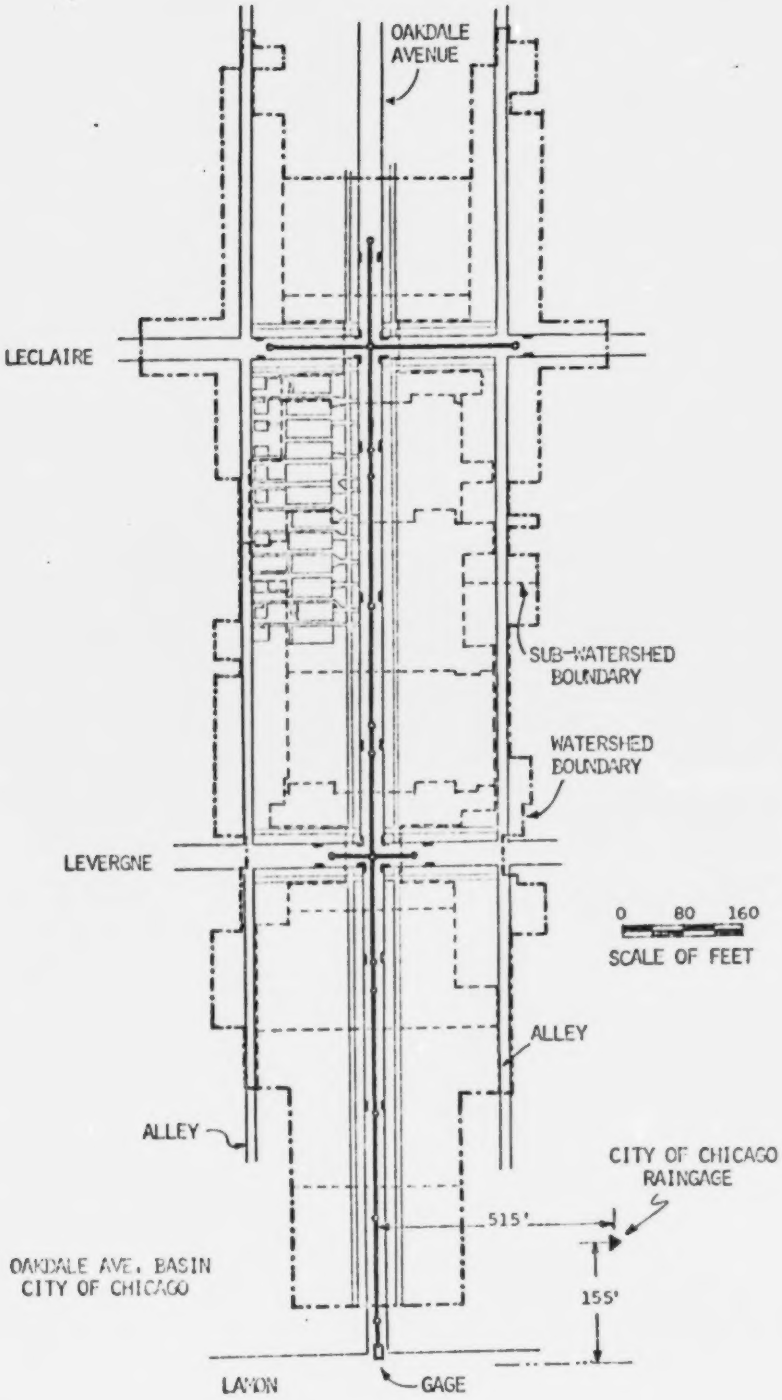

OAKDALE AVENUE
LECLAIRE
SUB-WATERSHED BOUNDARY
WATERSHED BOUNDARY
LEVERGNE
0 80 160
SCALE OF FEET
ALLEY
ALLEY
CITY OF CHICAGO RAINGAGE
515'
155'
OAKDALE AVE. BASIN
CITY OF CHICAGO
LAMON
GAGE

Figure 8-3. PLAN OF 12.9-ACRE OAKDALE AVENUE DRAINAGE BASIN, CHICAGO, ILLINOIS.

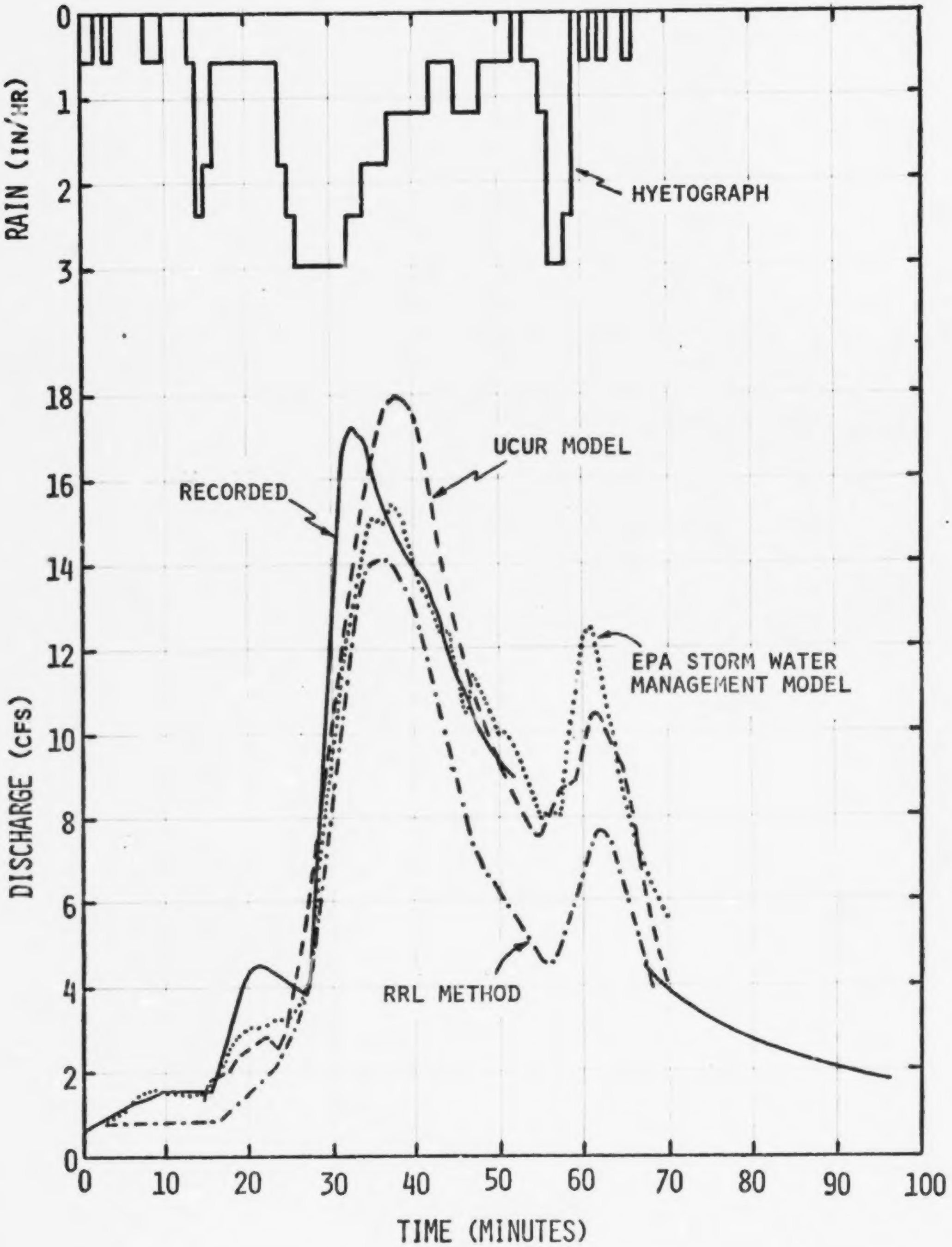


Figure 8-4. RAINFALL HYETOGRAPH, CALCULATED AND OBSERVED RUNOFF HYDROGRAPHS. OAKDALE AVENUE BASIN, CHICAGO. SECOND STORM OF JULY 2, 1960.

UCUR Model (18.1 cfs) was closer to the recorded peak (17.4 cfs) probably because the model considered a reduced infiltration capacity due to the wet antecedent condition of the soil caused by the previous rain which occurred earlier the same day. Therefore, some overland flow was generated from the pervious areas and contributed to the first peak. In comparison, the RRL Method without taking into consideration flow from pervious surfaces calculated lower peaks. The three methods gave a comparable first rising limb. However, the RRL Method calculated a lower recession than that obtained by the other two methods, producing a smaller total runoff than that recorded.

For the storm of July 7, 1964, the runoff hydrograph at the outlet of the Oakdale Avenue Drainage Basin was calculated by three methods: the UCUR Model, the RRL Method and the Chicago Method. The resulting hydrographs are plotted in Figure 8-5 together with the recorded hydrograph and the 86-minute storm hyetograph. The two peaks predicted by the UCUR Model and the Chicago Method occurred at about the same time, i.e., around 17 and 36 minutes after the beginning of the storm. The two peaks predicted by the RRL Method were both delayed for about three minutes. The recorded peaks occurred 20 and 37 minutes after the beginning of the storm. Therefore, the three methods agreed quite satisfactorily in the timing of the peaks. The UCUR Model, the RRL Method and the Chicago Method calculated a first peak of 3.9 cfs, 6.0 cfs and 6.0 cfs respectively, and a second peak of 8.8 cfs, 11.4 cfs and 12.1 cfs respectively. Again, the peaks predicted by the UCUR Model (3.9 cfs and 8.8 cfs) were closer to the recorded peaks (4.2 cfs and 9.6 cfs). By using the UCUR Model, the storm under consideration generated a very small amount of overland flow from pervious areas 30 minutes after the beginning of the rainfall and for 10 minutes only. Since the RRL Method does not take into consideration flow from pervious areas and since the overland flow from pervious areas predicted by the UCUR Model was very small, a better agreement between these two methods was expected. The three methods calculated comparable rising limbs for both peaks. However, the two recessions calculated by the UCUR Model were lower than the recessions predicted by the other two methods.

Bloody Run, Cincinnati. The Bloody Run Sewer Watershed in Cincinnati, Ohio was selected to test the EPA Storm Water Management Model and the UCUR Model. The Bloody Run Sewer Watershed is an urban drainage area comprised of 2380 acres of rolling terrain and drained by a combined sewer system designed for a 10-year frequency. This watershed has an average population of about 12 persons per acre and about 45% of its area is pervious (see Sections 3 and 4, Vol. II).

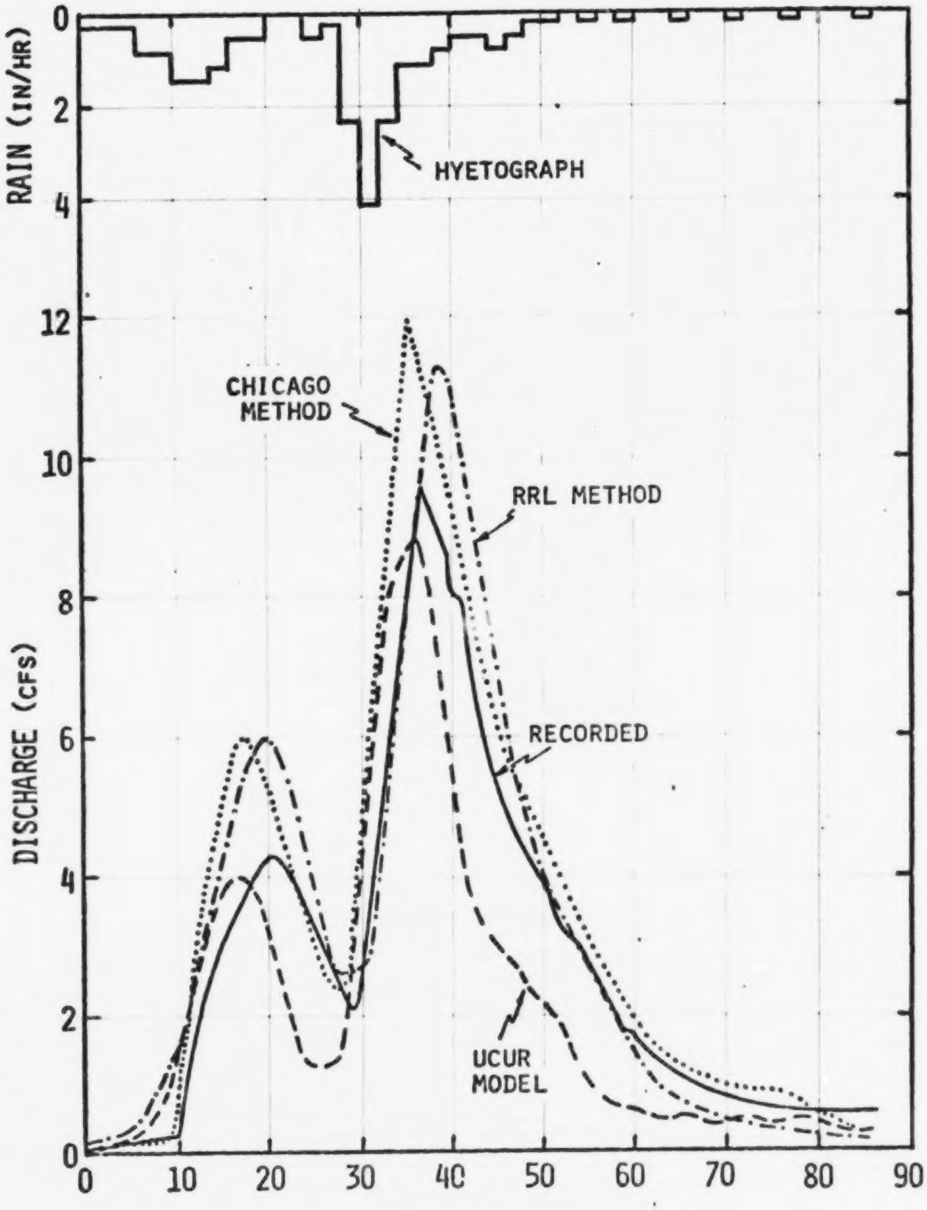


Figure 8-5. RAINFALL HYETOGRAPH, CALCULATED AND OBSERVED RUNOFF HYDROGRAPHS. OAKDALE AVENUE BASIN, CHICAGO. STORM OF JULY 7, 1964.

Data compilation from three years of continuous recording of rainfall in three locations and runoff quantity and quality in five locations within the watershed was carried out by the University of Cincinnati. The Bloody Run Sewer Watershed presented an excellent opportunity for testing the EPA SWMM and the UCUR Model under conditions of rough discretization because of its large area in comparison with other test sites and because of the detailed data available.

In order to use the EPA SWMM, the Bloody Run Sewer Watershed was divided into 38 subcatchments which were further subdivided into 71 subareas. The model used 117 pipe elements including manholes and 35 gutter/pipe elements. Testing was also conducted without gutter/pipe elements. Five minute intervals were used in the computations.

In order to apply the UCUR Model, the Bloody Run Sewer Watershed was divided into 37 subcatchments which were further subdivided into 122 subareas. The flow from these subareas was conducted into 90 main gutters and from there into 37 sewer inlets for routing through the sewer system. The sewer system was represented by 36 pipes ranging in size from 3 ft in diameter to rectangular 10' x 15'. Computations were performed in five minute intervals.

Information concerning the Bloody Run sewer system (shape, slope and length of sewer pipes) and the watershed perviousness, ground surface slopes, gutter lengths, etc.) were obtained from sewer maps, topographic maps and aerial photographs (Sections 3 and 4, Vol. II).

Two rainstorm events which occurred on November 9, 1970, and May 13, 1971, are examined herein. The EPA Storm Water Management Model received as input three rainfall hyetographs for each event obtained from the three raingages which cover the Bloody Run Watershed. Each of these rainfall hyetographs was associated with its corresponding subcatchments. The UCUR Model received as input an average rainfall hyetograph for each event obtained by using the Thiessen Method (Figures 8-6, 8-7). The portions of the Bloody Run Watershed which correspond to each of the three raingages are: Laidlaw Gage, 8.2%; Woodward Gage, 63.9%; and Ridge Gage, 27.9%.

The two cases presented are not intended to be comparative since different values for the constants of Horton's equation were used in the two models to simulate infiltration. The default values of the infiltration constants were used in the EPA SWMM, i.e., initial infiltration capacity of 3.00 inches per hour, constant infiltration

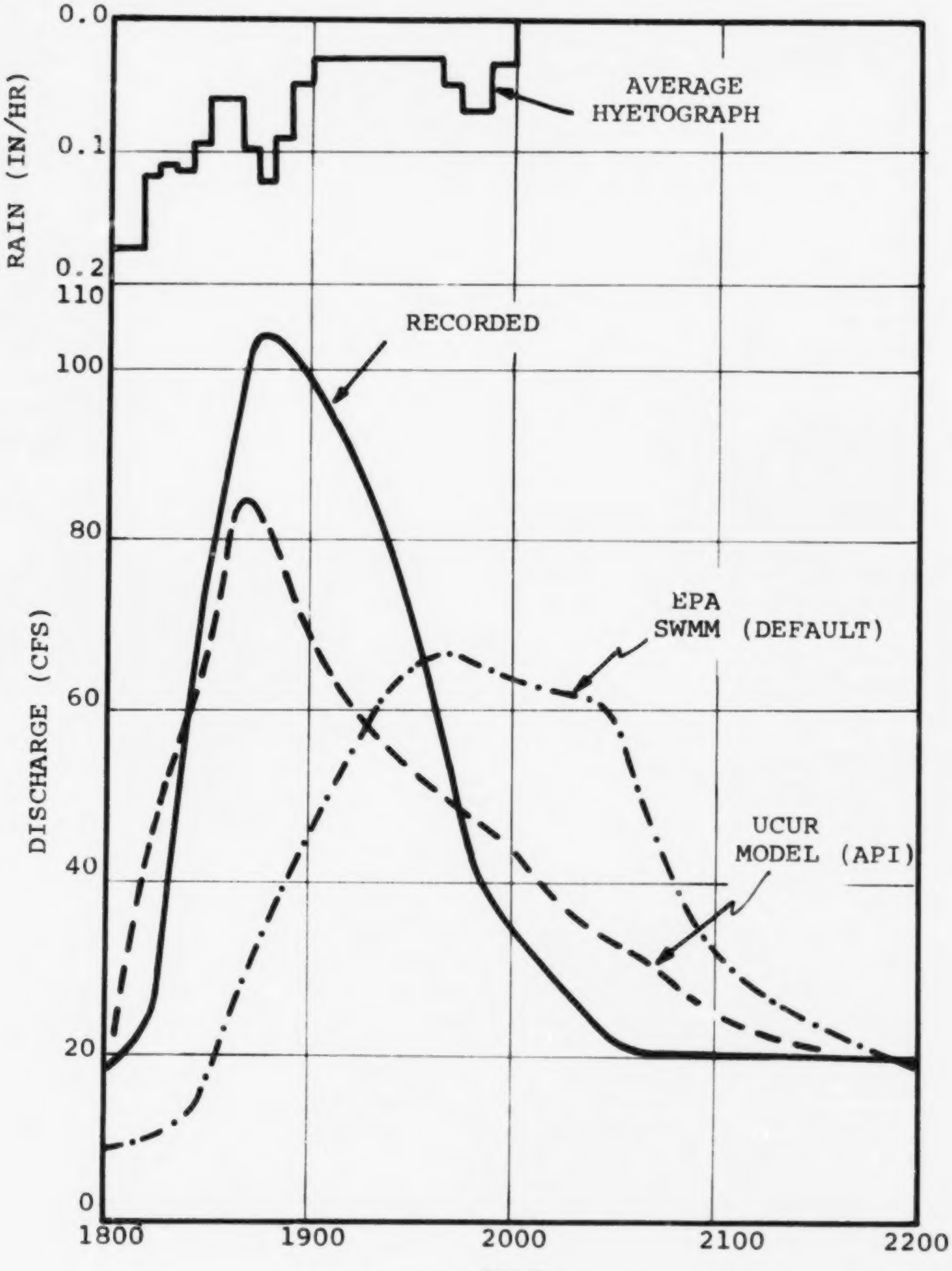


Figure 8-6. CALCULATED AND RECORDED RUNOFF HYDROGRAPHS. STORM OF NOVERMBER 9, 1970. OUTLET OF BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

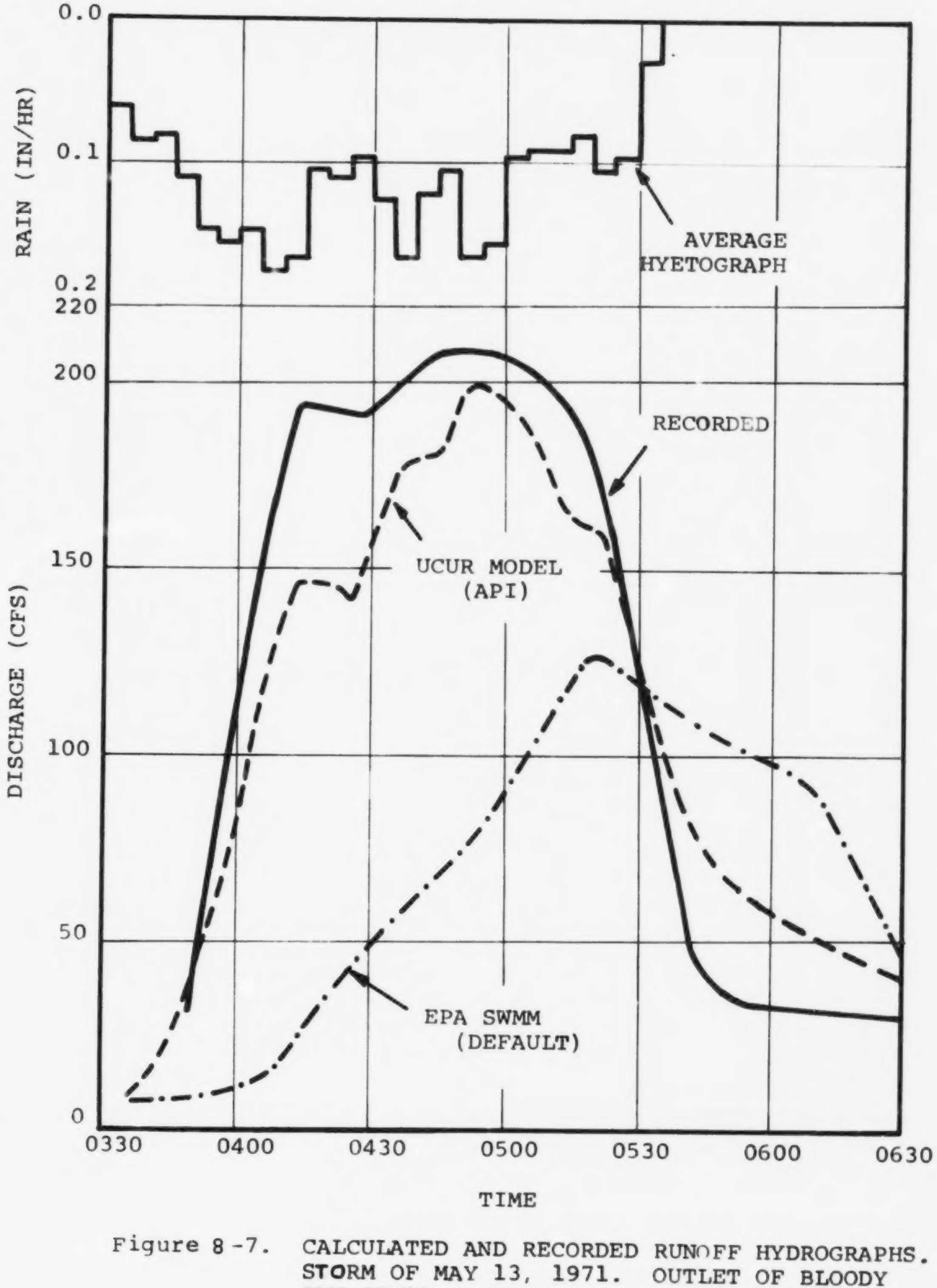


Figure 8-7. CALCULATED AND RECORDED RUNOFF HYDROGRAPHS. STORM OF MAY 13, 1971. OUTLET OF BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

capacity of 0.52 inches per hour and decay rate of infiltration 0.00115 sec^{-1}. The UCUR Model used infiltration values related to the Antecedent Precipitation Index (Section 4) which were found to be substantially lower than the default values. The intention was to demonstrate the influence of the selected infiltration input parameters on the simulated outlet hydrographs. Further studies using the same parameters for both models were unfortunately prevented because of budget limitations. For a series of rigorous comparisons reference [6] is recommended.

The UCUR Model generated hydrographs close to those recorded for both rains (Figures 8-6, 8-7). The EPA SWMM calculated lower and delayed in time peaks for both events as it was expected since higher values of Horton's infiltration constants were used as input. The hydrographs obtained by the EPA SWMM were not as sensitive to changes in the number of gutter pipe elements being used.

The UCUR Model required approximately 50% lower execution time than the EPA SWMM in all cases studied. The IBM 360/65 computer of the University of Cincinnati was used.

Considering the detailed approach used and the dynamic capability of the UCUR Model, it is believed that simulation would improve if individual hyetographs with their associated catchments were used as input instead of an average Thiessen hyetograph.

CONCLUSIONS

A brief outline of various methods for the calculation of urban **stormwater** runoff hydrographs has been presented chronologically following the evolution of the mathematical modeling art as used in the field of sewer system design and analysis.

More recent methods, such as the Chicago Method, the Road Research Laboratory (RRL) Method, the University of Cincinnati Urban Runoff (UCUR) Model and the Environmental Protection Agency Storm Water Management Model (EPA SWMM) were tested and compared in two case studies involving small urban watersheds (Chicago 10-acre tract, and Oakdale 12.9-acre drainage basin). All methods successfully reproduced the corresponding storm water runoff hydrographs.

The UCUR Model and the EPA SWMM were tested in a large urban watershed of diversified land uses (Bloody Run Sewer Watershed, Cincinnati, Ohio, 2380 acres). Discrepancies between the computed and recorded hydrographs indicated that the estimation of the infiltration parameters to be used as input in such models is of vital importance in the case of large urban watersheds. The EPA SWMM used its default values of Horton's infiltration parameters. The UCUR Model, although simpler in structure than the EPA SWMM, provided better results because it used values of infiltration constants obtained from the Antecedent Precipitation Index of each rain in connection with infiltrometer tests conducted in the Bloody Run Area (see Section 4).

Further improvement of methods for predicting urban stormwater runoff hydrographs can be carried out but the accuracy of input parameters should be considered to be of equal or perhaps greater importance.

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SECTION 9

ANALYSIS OF THE BLOODY RUN SEWER OVERFLOW DATA

INTRODUCTION

The purpose of this investigation was to study the overflow in Bloody Run Sewer Basin for a period of one year and to investigate a water balance to more fully understand what happens to water that enters a drainage area as rain.

The description of the Bloody Run Sewer Watershed is given in Section 3, Vol. II of this report. The description and layout of Bloody Run Sewer and monitoring system is given in Section 4, Vol. II of this report.

During 1952 the Department of Sewers of the City of Cincinnati estimated the average dry weather flow at the outlet of the Bloody Run Combined Sewer System as 6.81 mgd. The 30 inch diameter interceptor at the outlet of the Bloody Run Sewer System conducts the sewage to the Mill Creek Waste Treatment Plant in Cincinnati. The regulating structure at the inlet to the interceptor has been set to allow a maximum flow into the interceptor of 3 times the average dry weather flow, i.e. 20.43 mgd or 31.6 cfs. The reason for setting the regulator at 3 times the average dry weather flow was because the daily peak sanitary flow was approximately three times the average dry weather flow.

Table 9-1 provides data concerning overflows which occured during the period from August 19, 1970 to August 4, 1971, representing a complete one-year record of overflows. The volume of water which overflowed during each rain is provided. The total volume of runoff from the watershed during each rain is calculated by adding the volume of water intercepted to the volume of water which overflowed.

The total volume of runoff from the Bloody Run Watershed during the monitored rainstorms is valuable information since by estimating from the rainfall records the total volume of water precipitated, the volume of the water losses (i.e. infiltration, evaporation, transpiration, etc.) may be obtained. The plot of the total volume of runoff generated by each individual rainfall versus the accumulative probability frequency distribution appears to be a straight line on logarithmic-probability paper as shown in Figure 9-1.

Table 9-2 gives the maximum monthly discharges, in cfs, recorded at the outlet of the Bloody Run Sewer Watershed

Table 9-1. BLOODY RUN SEWER WATERSHED OVERFLOW DATA.

Date	Average Rainfall Duration (min.)	Average Depth of Precipitation (inches)	Time of Overflow		Time Since Last Overflow (hours)	Duration of Overflow (min.)	Volume of Overflow $(ft^3 x 10^4)$	Total Vol. of Runoff $(ft^3 x 10^4)$	No. of Peaks	Max. Peak (cfs)	Time of Max. Peak
			Start	End							
8/19/70	30	0.17	1403	1523		80	49.0	64.2	1	202	1415
8/20/70	26	0.42	2005	2139	29	94	62.1	73.9	2	216	2026
8/22/70	65	0.79	1706	1929	43	143	94.3	112.3	1	216	1730
9/3/70	115	0.40	1805	2042	287	157	47.0	76.8	3	167	1938
9/5/70	45	0.07	0117	0159	29	42	16.1	24.1	1	134	0122
9/8/70	55	0.44	1648	1828	87	80	47.5	62.7	1	121	-
9/13/70	31	0.99	2232	0058	124	146	56.7	84.4	2	140	2052
9/21/70	10	0.05	1545	1612	183	27	9.7	14.8	1	110	1552
9/22/70	20	0.49	2131	2235	29	84	31.4	47.3	1	125	2145
9/24/70	30	0.23	1817	1936	43	79	27.0	42.0	2	118	1845
9/25/70	151	1.13	0732	1145	12	253	76.6	124.6	2	103	0745
9/26/70	80	0.54	1513	1720	27	127	42.6	66.7	1	133	1545
9/26 & 27/70	175	0.19	2150	0020	5	150	33.1	51.9	1	98	2250
9/27/70	240	0.12	0315	0445	3	90	3.0	11.2	1	43	0130
10/9 & 10/70	234	0.36	2340	0303	307	203	17.8	56.2	4	75	2352
10/12/70	803	0.31	2040	2356	66	196	180.0	217.2	5	642	2200
10/13/70	160	0.18	0150	0345	2	115	72.7	94.5	2	370	0252
10/14/70	300	0.44	0344	0740	24	236	194.9	239.6	3	568	0430
10/20/70	344	0.73	0457	1022	141	325	227.6	239.2	4	252	0607
10/20/70	105	0.06	1220	1429	2	129	24.0	43.4	1	200	1230
10/20/70	41	0.06	1601	1706	2	65	27.3	39.6	1	202	1615
10/20/70	202	0.16	1753	2036	1	163	59.2	90.1	1	200	1837
10/29/70	326	0.07	0842	0933	204	57	7.8	18.6	1	70	0905
10/29/70	31	0.05	1239	1306	3	27	.3	5.4	1	38	1300
10/29/70	116	0.11	1412	1558	1	106	15.5	35.6	2	90	1522
11/2/70	66	0.23	0520	0656	85	96	48.6	66.8	1	642	0552
11/2/70	133	0.32	0853	2120	2	742	178.3	319.0	2	503	1030
11/9/70	78	0.11	1820	2006	165	106	29.0	49.1	1	121	1838
11/14/70	559	0.68	0844	1712	109	508	94.4	190.7	2	-	-
11/19/70	32	0.10	2227	2306	125	39	12.4	20.0	1	123	2245
11/20/70	250	0.76	0035	0512	2	277	372.1	424.6	4	515	0230
11/20/70	29	0.08	0803	0858	3	55	9.1	19.5	2	107	0830

Table 9-1. BLOODY RUN SEWER WATERSHED OVERFLOW DATE (continued)

Date	Average Rainfall Duration (min.)	Average Depth of Precipitation (inches)	Time Start of Overflow	Time End of Overflow	Time Since Last Overflow (hours)	Duration of Overflow (min.)	Volume of Overflow (ft³x10⁴)	Total Vol. of Runoff (ft³x10⁴)	No. of Peaks	Max. Peak (cfs)	Time of Max. Peak
12/3/70	15	0.13	2119	2204	327	45	33.0	41.5	1	259	2137
12/11/70	15	0.08	1335	1422	184	47	22.1	31.0	1	178	1353
12/11/70	195	0.28	1607	2000	4	113	24.7	46.1	1	567	1845
12/12/70	365	0.54	0102	0608	5	306	318.9	376.9	4	414	0145
12/16/70	215	0.27	0555	0911	96	196	69.6	106.7	2	240	0800
12/16/70	214	0.30	1040	1413	1	213	178.0	218.3	1	316	1130
12/16 & 17/70	338	0.57	1623	0035	4	372	335.6	456.0	3	367	2045
12/21/70	286	0.38	0625	1022	103	237	140.0	184.9	2	232	0825
12/21 & 22/70	420	0.74	2154	0619	12	505	616.0	711.6	3	475	0045
12/23/70	193	0.34	0223	0721	20	298	223.3	279.8	3	331	0408
1/3 & 4/71	-	-	1900	0730	283	750	435.6	578.0	-	-	-
1/13/71	213	0.31	1138	1453	220	195	131.1	170.1	2	270	1215
1/13/71	223	0.26	1525	1905	1	220	105.1	146.8	2	207	1815
1/14/71	145	0.34	0139	0512	7	214	231.4	272.0	3	429	0203
2/4/71	485	0.69	0252	1649	502	837	754.3	912.9	1	697	0623
2/4 & 5/71	128	0.15	2324	0140	7	136	76.6	102.4	1	246	2345
2/5/71	21	0.17	0321	0530	2	229	126.6	151.1	1	521	0341
2/11/71	Snowmelt	0.23	1522	1730	164	128	1.9	26.1	1	33	1615
2/12/71	"	0.10	0757	1623	14	506	57.8	153.7	3	89	1115
2/12/71	"	0.27	1740	2100	1	200	161.8	139.7	2	285	1938
2/12 & 13/71	"	0.20	2147	0035	1	163	50.8	82.6	1	129	2308
2/17/71	81	0.13	0625	1645	102	620	213.1	330.6	3	267	0645
2/19/71	140	0.25	1928	2236	51	188	161.4	187.0	2	371	2015
2/21,22 & 23/71	1149	2.52	2125	0045	47	1540	2907.0	3217.9	6	871	0052
2/26/71	65	0.06	0617	0644	78	27	1.0	6.1	1	43	0630
2/26/71	93	0.07	0746	0859	1	73	12.5	16.2	1	89	0808
2/26/71	38	0.06	1645	1803	8	78	30.1	44.8	1	142	1730
3/6/71	93	0.21	0952	1103	220	71	60.3	73.7	1	329	1011
3/6/71	375	0.24	1120	1930	0	490	393.3	486.2	2	416	1222
3/10/71	Snowmelt	0.21	0539	0819	58	150	29.7	57.1	2	99	0723

Table 9-1. BLOODY RUN SEWER WATERSHED OVERFLOW DATA (continued)

Date	Average Rainfall Duration (min.)	Average Depth of Precipitation (inches)	Time Start of Overflow	Time End of Overflow	Time Since Last Overflow (hours)	Duration of Overflow (min.)	Volume of Overflow (ft^3x10^4)	Total Vol. of Runoff (ft^3x10^4)	No. of Peaks	Max. Peak (cfs)	Time of Max. Peak
3/10/71	Snowmelt	0.40	0839	1807	0	568	333.7	441.2	1	222	1045
3/19/71	"	0.31	0509	0712	203	123	91.8	115.1	2	295	0538
4/1/71	148	0.10	1453	1515	320	22	0.9	5.0	1	42	1500
4/1/71	153	0.22	1712	1952	2	160	90.3	120.6	3	223	1911
4/1/71	170	0.12	2002	2257	0	175	120.2	153.3	2	310	2122
4/6/71	40	0.09	0428	0441	126	13	0.2	2.7	1	37	0430
4/6/71	28	0.02	0447	0524	0	37	3.9	10.9	1	60	0507
4/13/71	278	0.25	1816	2109	181	172	101.6	134.2	2	232	2015
4/27/71	68	0.47	1922	2039	24	76	124.3	138.7	1	762	1942
5/2/71	80	0.06	0438	0457	122	19	0.3	3.9	1	34	0452
5/2/71	90	0.09	0552	0639	1	47	11.3	20.2	1	94	0615
5/5/71	55	0.17	2233	2348	88	75	68.1	32.3	1	308	2300
5/6/71	95	0.09	0520	0554	6	34	6.8	13.2	1	88	0530
5/6/71	103	0.39	0709	0922	1	133	232.5	257.7	1	744	0737
5/6/71	45	0.07	1433	1518	5	45	6.4	14.9	1	76	1500
5/7/71	530	1.11	1319	2211	22	532	883.7	984.5	4	677	1845
5/13/71	162	0.23	0349	0520	126	91	94.2	115.8	1	209	0445
5/13/71	254	0.12	0949	1054	4	65	5.3	17.6	1	52	1015
5/19/71	33	0.15	2052	2140	154	48	53.7	62.8	1	418	2100
5/24 & 25/71	280	0.62	2318	0337	122	259	430.1	479.1	3	610	0015
6/7/71	35	0.09	0925	0958	293	33	8.6	14.9	1	106	0835
6/7/71	35	0.27	1605	1644	7	39	26.7	34.1	1	202	1615
6/14/71	84	0.87	1603	1745	167	102	253.9	273.2	1	1082	1611
6/15/71	148	0.25	1056	1253	17	117	94.2	116.4	1	314	1141
6/19/71	165	0.32	1200	1530	95	210	96.0	143.3	3	252	1210
6/28/71	60	1.10	1310	1500	214	110	285.4	318.0	1	1215	1322
7/10/71	50	1.75	1430	1730	287	180	590.2	630.0	1	1450	1452
7/11/71	15	0.18	0120	0155	7	35	28.4	39.8	1	252	0122
7/11/71	35	0.18	1150	1345	10	115	94.3	155.5	1	358	1200

Table 9-1. BLOODY RUN SEWER WATERSHED OVERFLOW DATA (continued)

Date	Average Rainfall Duration (min.)	Average Depth of Precipitation (inches)	Time Start of Overflow	Time End of Overflow	Time Since Last Overflow (hours)	Duration of Overflow (min.)	Volume of Overflow ($ft^3 x 10^4$)	Total Vol. of Runoff ($ft^3 x 10^4$)	No. of Peaks	Max. Peak (cfs)	Time of Max. Peak
7/14/71	25	0.20	0035	0150	59	75	110.0	138.5	1	650	0045
7/18/71	22	0.07	1340	1422	108	44	6.8	21.0	1	96	1400
7/19/71	80	0.25	1540	1800	1	140	106.6	135.0	1	348	1615
7/19/71	25	0.10	0335	0410	9	35	20.9	32.2	1	200	0350
7/23/71	15	0.15	1905	2000	111	55	53.8	65.2	1	381	1915
7/24/71	600	0.65	0600	1230	10	390	106.0	180.0	5	282	1045
7/28/71	90	0.51	1850	2100	102	130	204.6	269.0	2	603	1852
8/ 3/71	75	0.25	1105	1330	134	145	113.7	170.9	2	410	1145
8/ 4/71	160	0.20	0120	0310	13	110	103.7	150.2	2	142	0200
8/ 4/71	150	0.37	1030	1310	7	160	196.6	231.0	2	325	1100

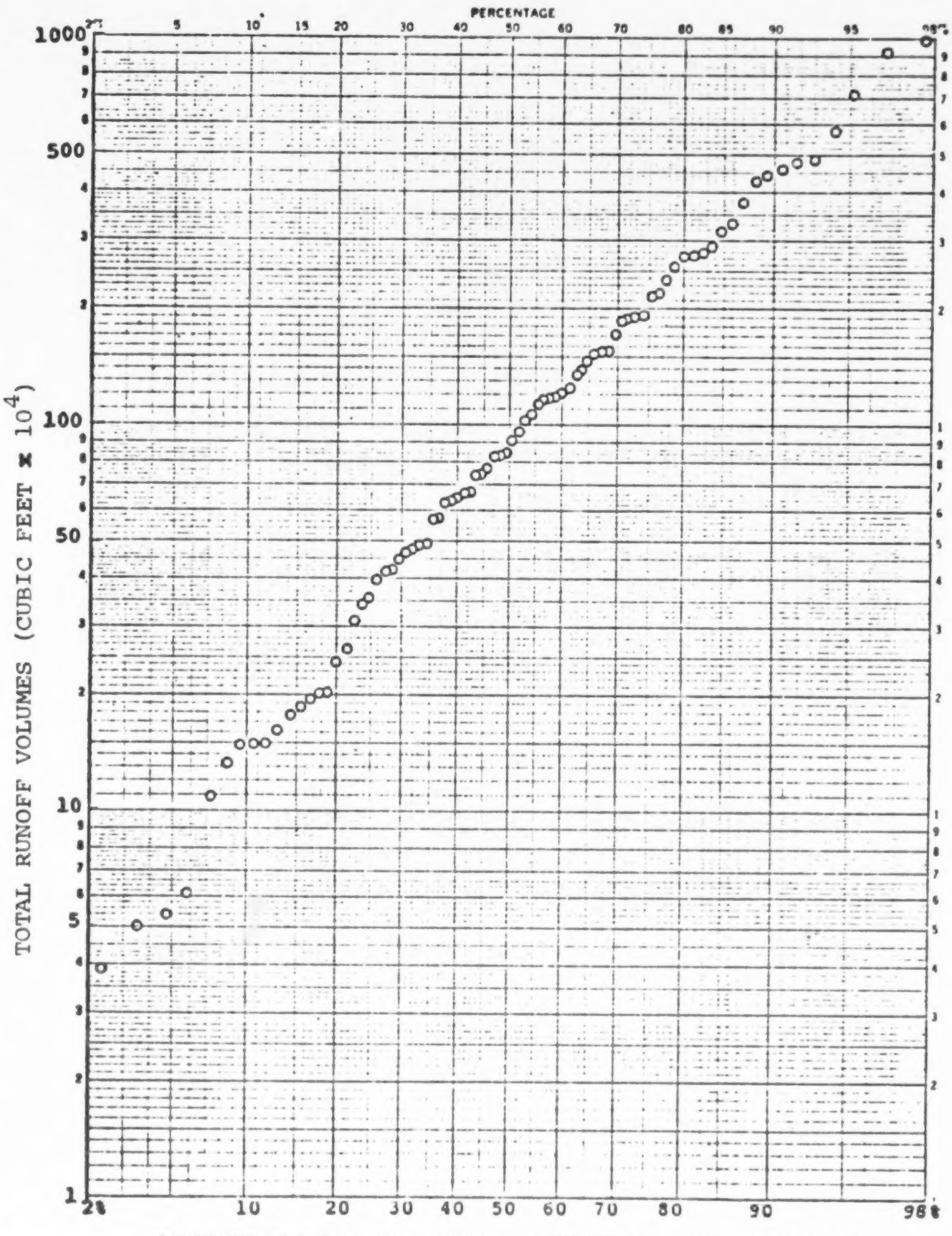


Figure 9-1. ACCUMULATIVE PROBABILITY FREQUENCY DISTRIBUTION OF TOTAL RUNOFF VOLUMES

Table 9-2. RETURN PERIOD OF MONTHLY MAXIMUM DISCHARGES. BLOODY RUN WATERSHED

Month	Monthly Max. Discharge (cfs)	Rank m	Return Period, T
Oct. 1970	642	12	1.83
Nov. 1970	642	13	1.69
Dec. 1970	414	19	1.16
Jan. 1971	428	17	1.29
Feb. 1971	871	6	3.67
Mar. 1971	416	18	1.22
Apr. 1971	762	9	2.44
May 1971	744	10	2.20
June 1971	1215	2	11.00
July 1971	1450	1	22.00
Aug. 1971	778	8	2.75
Sept. 1971	950	4	5.50
Oct. 1971	691	11	2.00
Nov. 1971	557	14	1.57
Dec. 1971	508	16	1.38
Jan. 1972	178	21	1.05
Feb. 1972	266	20	1.10
Mar. 1972	528	15	1.47
Apr. 1972	823	7	3.14
May 1972	975	3	7.35
June 1972	939	5	4.40

(upstream from the interceptor) in chronological order from October 1970 to June 1972. All discharges are ordered according to their magnitude, assigning the number one to the overall largest discharge. The return period T of each discharge is computed as $T = (1+n)/m$, where $n = 21$ is the number of discharges in the record and m is the rank of each particular discharge. The monthly maximum peak discharges plot as a straight line versus their corresponding return periods in skewed probability paper for plotting extreme values, as illustrated in Figure 9-2.

WATER BALANCE

In order to more fully understand what happens to the water that enters a drainage area as rain, a water balance for the Bloody Run area was investigated.

Notable variables in the water balance were: total volume of rainfall, total volume of stream flow, evaporation, transpiration, storage, and lateral flow. For a graphical illustration of water balance, refer to Figure 9-3.

Expressed as a mathematical equation:

$$X = Y + Z + W + U + I$$

where X = total rainfall in cubic feet

Y = total stream flow in cubic feet

Z = evaporation and transpiration in cubic feet

W = lateral flow, below ground, in cubic feet

U = storage in cubic feet

I = infiltration in cubic feet

For the Bloody Run area, the total rainfall as recorded by the ABBE Observatory in Cincinnati was 37.45 inches for the period from August 19, 1970 to August 19, 1971 [1]. Converting this to cubic feet gave:

$$\frac{37.45''}{12} \times 2380 \text{ acres} \times 43560 \frac{\text{ft}^2}{\text{acre}} = 323{,}550{,}000 \text{ ft}^3$$

The 323,546,000 ft^3 represented the total amount of water precipitated on the drainage basin during the specified period of time.

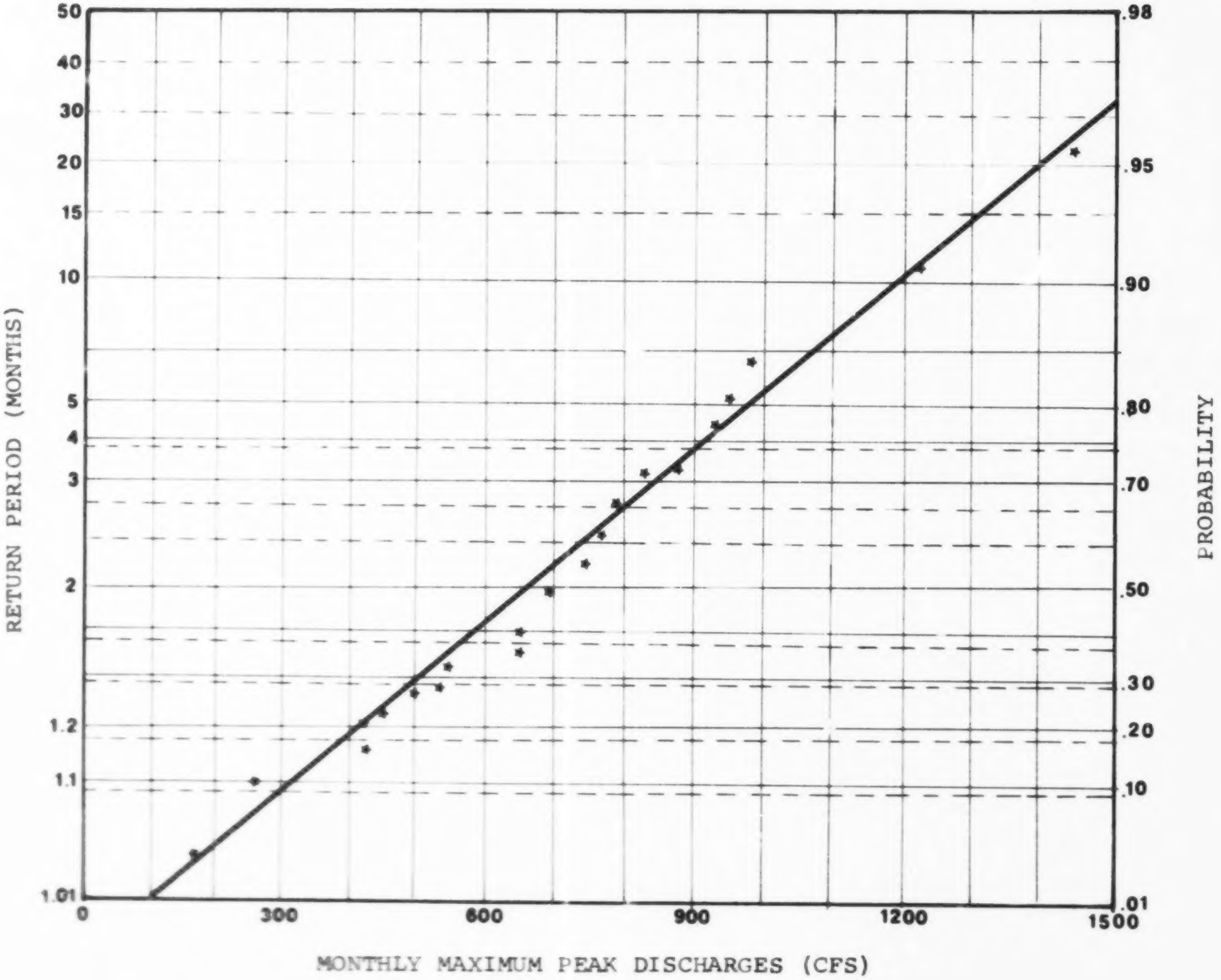


Figure 9 -2. RETURN PERIOD OF MONTHLY MAXIMUM PEAK DISCHARGES

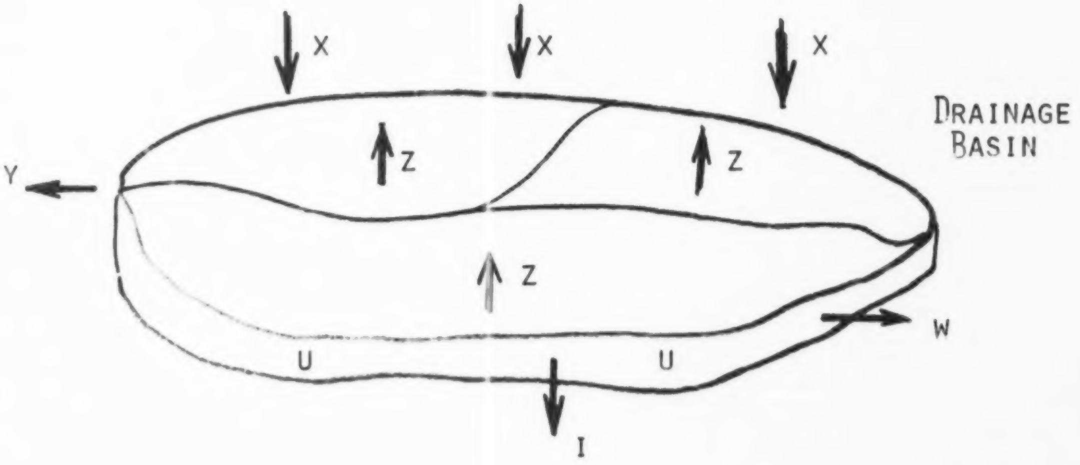

X
X
X
DRAINAGE BASIN
Y
Z
Z
Z
W
U
U
I

Figure 9-3. SCHEMATIC WATER BALANCE FOR A DRAINAGE BASIN.

The total volume of water which ran off of the watershed and was recorded at the sewer outlet of the Bloody Run Sewer System (Table 9-1) during the one year period was 190,300,000. This leaves:

$$\begin{array}{r} 323{,}550{,}000 \\ \underline{190{,}300{,}000} \\ 133{,}250{,}000 \end{array} \text{ ft}^3$$

This remainder of 133,250,000 ft^3 consisted of infiltration, evaporation, transpiration, storage and lateral flow.

By using Figures 52 and 53 on pages 132 and 133 of the Hydrology Handbook by the American Society of Civil Engineers [2], values for evaporation including interception and transpiration were estimated. These figures were developed for the Midwest region of the U.S.A. Monthly temperature and rainfall records were needed for using Figures 52 and 53. As suggested in the Hydrology Handbook, a correction factor of 0.80 was used to reduce evaporation losses in order to reflect local conditions of the Bloody Run Watershed, i.e. sandy soil, rolling topography, high humidity etc. The evaporation and transpiration estimates are shown in Table 9-3.

Not all land in the Bloody Run Watershed is covered by vegetative growth. Roughly 20% of the land area is open and 50% is residential. The remainder is industrial and commercial land, so evaporation and transpiration may be assumed negligible in these non-vegetated areas. If one half of the residential land is assumed to be open, i.e. is not covered by homes and roads, then the total available open land would be 20% + (50%)/2. This is almost 50% of the total area or 1070 acres.

Evaporation and interception would therefore account for:

$$\frac{12.50''}{12} \times 1070 \text{ acres} \times 43{,}560 \; \frac{\text{ft}^2}{\text{acre}} = 48{,}550{,}000 \text{ ft}^3$$

Transpiration would account for:

$$\frac{12.40''}{12} \times 1070 \text{ acres} \times 43{,}560 \; \frac{\text{ft}^2}{\text{acre}} = 48{,}150{,}000 \text{ ft}^3$$

The total water available for storage, infiltration, and lateral flow would be:

$$133{,}250{,}000 - 48{,}550{,}000 - 48{,}150{,}000 = 36{,}550{,}000 \text{ ft}^3$$

Table 9-3. EVAPORATION AND TRANSPIRATION ESTIMATES

Month	Rainfall (inches)	Mean Temperature	Evaporation (inches, corrected)	Transpiration (inches)
August '70	0.89*	75	0.50*	0.50*
September	3.55	73	1.50	2.30
October	2.54	58	0.85	1.30
November	2.00	44	0.60	0.10
December	3.55	37	0.65	-
January '71	2.47	28	0.55	-
February	5.89	33	1.05	-
March	2.55	40	0.80	-
April	1.04	53	0.50	1.10
May	3.31	61	1.35	1.60
June	5.18	76	2.05	2.70
July	2.85	74	1.45	2.50
August '71	1.63**	73	0.65**	0.30**
TOTAL	37.45		12.50	12.40

* For the last 10 days of August, 1970
** For the first 5 days of August, 1971

Further breakdown of the remaining water is almost impossible because it involves subsurface research.

Looking at the total picture, the water balance shows these approximate percentage relationships:

Runoff	59.0%
Evaporation	15.0%
Transpiration	14.5%
Storage, lateral flow and infiltration	11.5%

Much valuable information can be obtained from a water balance. It should be noticed that in this water balance about 60% of the water falling on the drainage basin as rain reached the sewer system as runoff. Knowing this could help in the design of sewers for newly developed areas. Also destruction of the natural ground cover, caused by the expansion of commercial and industrial areas will alter the values of evaporation and transpiration thereby causing an increase in the percentage of rainfall reaching the sewers.

The water balance is a valuable tool for evaluating the flow of water into and out of a drainage basin. As more water balances are investigated throughout the country, regional water balances can be developed and used in planning of sewer and drainage projects.

REFERENCES

Section 9

1. Local Climatological Data, U.S. Dept. of Commerce Nat'l. Oceanic and Atmospheric Administration, Environmental Data Service, Cincinnati, Ohio.

2. Hydrology Handbook, Manual of Engineering Practice No. 28, American Society of Civil Engineers, pp. 132-133.

SECTION 10

RETARDING BASIN CAPACITY DETERMINATION BASED ON BLOODY RUN SEWER OVERFLOW DATA

INTRODUCTION

Stormwater overflows at the outlets of combined sewers have been established as a definite source of pollution to receiving streams [1]. Although combined sewers are normally designed to carry flow from storm runoff many times greater than the average dry weather flow of the sewers, it is not feasible to transport the entire flow during heavy runoff periods to plants for treatment. Therefore, overflow devices are often installed at the outlets of the sewerage systems which divert the higher discharges directly to the receiving waters. The portion that is not diverted is transported in interceptor sewers to treatment facilities.

Since the nation as a whole has been rapidly becoming more aware of pollution and environmental problems, the interest in water pollution caused by combined sewer overflows has been rapidly increasing.

One of the methods of alleviating this problem is the use of retarding basins which store excess combined sewer flow, then release it back through the system at a rate that can be accepted by the interceptor sewers and treatment plants.

A method for stochastic design of retarding basins for storing combined sewer overflows has been developed by using normal statistical procedures utilizing short term measurements of overflow volumes. A method of checking the short term results with an estimate of the long term trends is also presented.

The design method is set forth on the assumption that storage is required in an existing combined sewerage system.

The result obtained by using this design procedure is a reasonable and adequate value of the design storage volume of a retarding basin based upon the requirements of a given sewerage system.

REVIEW OF THE LITERATURE

The problem of water pollution caused by combined sewer overflows is widespread. The American Public Works

Association Research Foundation published a report in 1967 for the United States Federal Water Pollution Control Administration (now the Environmental Protection Agency) concerning the problems of combined sewers [2, 3]. In the report, it was estimated that there are 1,329 jurisdictions in the United States with a population of 54 million served in whole or in part by combined sewers. An estimated 36 million persons are served entirely by combined sewers. There were 9,860 combined sewer overflows reported by 641 jurisdictions surveyed. In 1969 it was estimated that the cost of separating the Nation's combined sewers would be $49 billion including necessary plumbing changes on private property. Cost information was insufficient to get accurate estimates on alternative methods, however, they were estimated to range to $30 billion [4].

It has been shown by a number of investigators that storm runoff carries with it significant amounts of pollutants [5, 6, 7]. The American Public Works Association report states that, in general, urban runoff amounts to 1 percent of the raw sewage for a particular area [8]. The situation is aggravated by the increase in urbanization as is being experienced in the United States. With the increase in urbanization, there is an increase in impervious areas resulting in an increase in storm runoff [9]. In addition, increased urbanization increases the amount of pollutants available to be washed into the sewers by the runoff.

One alternative to complete separation of combined sewers is the use of retarding basins to store excess storm runoff with subsequent release into the sewerage system at a rate which can be handled by the treatment facilities. Storage of excess urban storm runoff is not new. A combined sewerage system including this concept of storage has been in operation at Columbus, Ohio since 1933 [10]. The system consists of three parts: intercepting sewers of sufficient capacity so that overflows are limited to a predetermined number each year; regulator chambers which control the flow into the interceptors; and storm stand-by tanks which give partial treatment to all storm sewage in the interceptors which is in excess of the amount that can be handled by the sewage treatment works. The tanks were designed to remove heavy solids and floating materials and also to act as detention tanks or retarding basins.

About 22 years ago, the City of Buffalo, New York made use of an old quarry for a retarding basin [11]. It was used basically for flood control but it did receive some sanitary sewage. In 1964 New York City designed a system using closed detention tanks in which excess combined sewer flows were stored, chlorinated, then released back through the treatment facilities when the storm flow had subsided [12].

The City of Chicago, Illinois has undertaken an unusual and enormous storage plan. The city's plans are to build a network of storage tunnels beneath the city to accept excess stormwater runoff [13, 14]. After the surface sewer storm flow has subsided, the tunnels will be evacuated and the stored waste water will be treated in Chicago's conventional treatment facilities. Boston, Massachusetts is considering a similar plan [15].

In Cleveland, Ohio a feasibility report was published in 1968 concerning a stabilization-retention basin in Lake Erie [16]. Here the idea is to construct a storage basin off shore in Lake Erie where overflows would receive partial treatment before being discharged into the lake.

The cities of Washington, D. C. and Sandusky, Ohio are using another new idea in storage. Large inflatable rubber containers will serve as storage facilities. In Washington, D. C., the rubber containers will rest on the bottom of the Anacostia River and in Sandusky they will be submerged in Sandusky Bay [17, 18]. However at the present time none is being used.

Chippewa Falls, Wisconsin and Springfield, Illinois have constructed and are currently operating open storage reservoirs [19,20]. At Chippewa Falls the combined sewer overflows are stored and when storm flows subside, they are pumped back to the interceptor for processing at the treatment plant. The operation in Springfield is somewhat different as the reservoir there is used for removal of suspended solids and partial removal of BOD. A minimum reservoir level is maintained and outflow is controlled by a notched weir which tends to stabilize the outflow.

The common methods of design of storage facilities are generally related to the rational formula, such as in the case of Springfield [20], or to hydrograph methods such as the modified Chicago hydrograph method as used in Cleveland [16]. Another method which is commonly used in Europe is one developed in Germany by Gunter, Muller, and Neuhaus [21]. This method uses an equation similar in form to the rational method plus some empirical curves from which the design volume of a retarding basin is found.

These current methods would appear to be somewhat inadequate since they do not take into consideration the stochastic nature of combined sewer overflows. Runoff and the associated overflows are related to rainfall and it is apparent that a correlative relationship should exist between these occurrences. However, Bottger and Van den Herik, from studies in The Netherlands, were not able to find useful relations between the total annual precipitation and the number of overflows per year. Neither did they establish correlation between precipitation and the volume of overflows per year, nor between the number of overflows per year and the volume of overflows per year [22]. Therefore, in further investigations into runoff-overflow relationships, it may be necessary to take other parameters into account.

Deterministic models such as the EPA Storm Water Management Model [24] and the University of Cincinnati Urban Runoff (UCUR) Model [23] are very useful tools in the design of sewerage systems and retarding basins; however, they lack the ability to simulate the stochastic nature of the problem without expensive computer runs for each of hundreds of different rainfalls in sequence as they would occur in nature.

DEVELOPMENT OF THE PROPOSED DESIGN METHOD

In the process of designing a reservoir or other water storage facility, the first consideration must be the determination of the design storage volume. After the volume is determined, the type of storage facility is decided, then the structural design is accomplished. Deviation from this sequence might be required when an optimization of benefit versus cost is desired.

The proposed design method set forth here is for the determination of the design storage volume. This method uses a stochastic approach to the solution of the problem of finding a reasonable and adequate value for the design storage volume. The word stochastic implies the existence of randomness in the variables being considered; here the overflow volumes and the required storage volumes are stochastic variables. Since the existence of random variables in the problem is acknowledged, it is necessary that the theory of probability be employed in the solution.

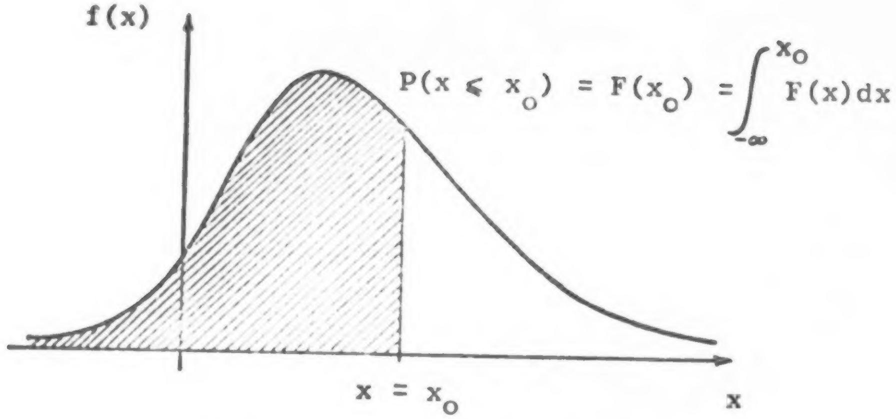
The relationship of probability to some continuous variable, such as overflow volume, is expressed by the probability density function, $f(x)$, and its integral the cumulative distribution function, $F(x)$. These two functions define the theoretical probability distributions of continuous random variables. It is emphasized that a specific value of a continuous random variable has no probability but only a density of probability. The cumulative distribution function provides the means to obtain a probability of a value occurring which is less than a given value, greater than a given value, or which is between two given values. The area under the density function and the maximum value of the cumulative distribution function must be unity. These relations are demonstrated graphically in Figure 10-1.

In determining the design storage volume, first the measured overflow volumes of each separate overflow event are considered. Deciding whether an overflow consists of one or more separate events may be somewhat arbitrary when, for example, the overflow actually stops for a short period of time and then starts again. In the strictest sense, any time that an overflow stops one event ends; and when overflow starts again, the next event begins no matter how short the time interval is between the two events. However, it may be convenient to choose a certain length of time during which there must be no overflow before each overflow is considered as a separate event. If the time between overflows is less than the chosen time interval, the volumes of the two overflows are added and considered as one overflow event. The individual designer is left with the responsibility of choosing the minimum time separating two events based on his engineering judgment and the characteristics of the individual project. Here, the minimum time between overflow events will be taken as one half of an hour with the time between overflows figured to the nearest whole hour.

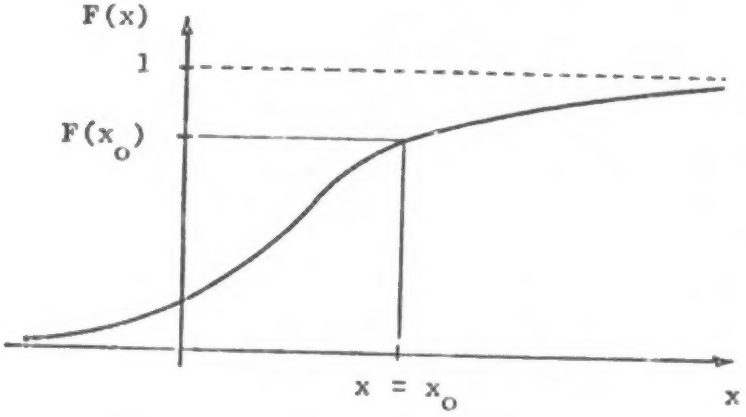
The measured overflow volumes of n events are ordered from smallest to largest where V_m is the overflow event volume and m is the order number (m = 1, 2, 3, ..., n). It is now necessary to fit the empirical distribution of overflow event volumes to some theoretical probability distribution. To do this, it is convenient to make use of probability paper which is designed so that the cumulative distribution function plots as a straight line. Each probability distribution has a different probability paper. If a probability paper for a distribution is not available, it may be constructed as described in reference [25] or [26].

If the data plotted on probability paper approximates a straight line, the empirical distribution may be approximated by the theoretical distribution. The degree to which the data approximates a straight line is a measure of the goodness of fit of the theoretical distribution to the empirical distribution.

There have been many methods of plotting data on probability paper. These methods are concerned with the position each value of the variate is plotted with respect to the



PROBABILITY DENSITY FUNCTION



CUMULATIVE DISTRIBUTION FUNCTION

Figure 10-1. TYPICAL PROBABILITY DENSITY FUNCTION, f(x) AND CUMULATIVE DISTRIBUTION FUNCTION, F(x)

probability scale which, on probability paper, is normally the abscissa. The plotting position m/(n + 1) has been recommended by Gumbel [25] and is used here.

The logarithmically transformed normal distribution,

$$f(x) = \frac{1}{\sigma_y \sqrt{2\pi}} \exp \left[- (y - \mu_y)^2 / 2 \sigma_y \right] \qquad (1)$$

where:
x is the variate

$y = \ln x$

μ_y is the mean of y, and

σ_y is the standard deviation of y

which is commonly called the lognormal distribution, has been used **later** in a case study of the Bloody Run Combined Sewer Watershed in Cincinnati, Ohio.

The overflow data, when plotted as described here, indicate that when an overflow occurs, there is some probability, P, that the overflow will be equal to or less than a given amount. To find the probability that the overflow will be equal to or greater than a given value, the probability that it will be less than the given value is subtracted from 1. This relationship is given below:

$$P\ (V > V_o) = 1 - P\ (V < V_o) \qquad (2)$$

where,
V is the overflow volume and

V_o is the given value.

From the overflow volumes, the storage volume required to prevent spilling from storage at the end of each overflow event is calculated. The required storage volume at the end of a specific overflow event is the volume of that specific overflow event plus the volume of the water remaining in storage as a result of previous events.

The volume remaining in storage from previous events at the end of a given event is calculated by subtracting the storage outflow rate times the time between the two overflows from the volume in storage at the end of the previous overflow event. However, this value cannot be less than zero.

The required volume of storage at the end of an overflow event is given by,

$$V'_c = V_c + (V'_{c-1} - \frac{dV}{dt} t), \text{ for } \frac{dV}{dt} t \leq V'_{c-1}$$

$$V'_c = V_c , \qquad \text{for } \frac{dV}{dt} t > V'_{c-1} \qquad (3)$$

where: V'_c = the required storage volume at the end of event c

V_c = the volume of overflow of event c

V'_{c-1} = the required storage volume at the end of event c-1

dV/dt = the rate of outflow from storage

t = the time between event c-1 and c

c = the chronological event order.

For the sake of simplification, the outflow rate dV/dt may be taken as a constant for a given storage facility. This has been done throughout the following and the constant value of dV/dt is represented by O, therefore, equation (3) may be rewritten as,

$$V'_c = V_c + (V'_{c-1} - Ot), \quad \text{for } Ot \leq V'_{c-1}$$

$$V'_c = V_c , \qquad \text{for } Ot > V'_{c-1} \qquad (4)$$

There is some variation in the method of measuring the values of t in equations (3) and (4). The general rule would be that t begins when outflow from storage begins and ends when outflow from storage is no longer possible. When an uncontrolled outlet from storage exists and it is possible for outflow to be continuous, the summation of all t's would equal the total length of time of observation. In this case, t would be measured from the end of one overflow event to the end of the succeeding overflow event. If the conditions are such that during overflows the interceptor sewer is flowing to capacity, then a control must be installed at the outlet of the storage basin which would prohibit outflows to the interceptor sewer during the time when the overflows are occurring. In this case the values of t would be measured from the end of one overflow event to the beginning of the next overflow event. The second case is the one used in the case study of the Bloody Run Sewer System Watershed.

The values of the required storage volumes at the end of overflow events, V', are now ordered in increasing magnitude as were the overflow volumes. The ordered required storage volumes are represented by V'_m , where m is the order number (m = 1, 2, 3, ..., n-1). The largest order number of the required storage volumes is one less than the number of overflows because the first overflow is used only to calculate the required storage volume at the end of the second overflow event. Thus, the required storage volume corresponding to the first overflow volume is lost because the previous overflow is unknown.

The required storage volume data may now be fit to some theoretical probability distribution. It is hypothesized that the required storage volumes will fit the same type of theoretical distribution as the overflow volumes but with different values of the distribution parameters, i.e., mean and standard deviation. The Bloody Run case study tends to verify this hypothesis. When an overflow occurs, there is a probability, $P(V' > V'_o)$, that the storage volume required to prevent spilling from storage, will be equal to or greater than some value, V'_o.

In the design of a retarding facility, the value of the outflow rate, O, may be a variable in the design problem. The value of O has a large effect on the magnitude of the storage volume required as it may be observed in the Bloody Run case study. All other variables being held constant, if the value of O is increased or decreased the storage volume required, corresponding to the same probability, will be decreased or increased. As O is increased and the smallest value of the quantity Ot, in equation (4), approaches the largest value of V'_{c-1} so that the expression in parenthesis on the right side of equation (4) in all cases approaches O, the distribution of the required storage volume, V', approaches the distribution of the overflow volume, V.

The relation of the value of O to the distribution, as indicated by its parameters including the mean, μ , and standard deviation, σ , may be found by repeating the procedure of finding the required storage volumes from the overflow volume and spacing data using equation (4) for several values of O. Each set of required storage volumes are then ordered from smallest to largest and plotted on the probability paper. The mean and standard deviation of each distribution is then found and plotted versus O. The curves obtained by this procedure will then indicate how the distribution varies with O.

The relationship of the required storage volumes, at a given probability, to O may also be desired. In this case, the distributions of the required storage volumes are found for various values of O, as was done above using equation (4). The required storage volumes of each of the distributions corresponding to the desired probability are then read from the plot and these volumes are plotted versus the values of O. This would be used if it had earlier been decided to store a given percentage of the total overflows.

To this point, consideration has been given only to the process of determining the distributions of the overflow volumes as they occur and the volume of storage required to prevent spilling at the end of an overflow. Some attention has also been given to how the required storage volume distribution will change with regard to the rate of outflow from the retention facility.

In the following, attention will be given to describing the data requirements and also to the generation of missing data. A method will also be introduced to generalize distributions which may have been determined from data obtained over short sampling periods.

DATA REQUIREMENTS

In order to find distributions that are truly representative of all overflows and storage volumes in the past and in the future, two basic conditions must be met. The first is that the characteristics of the watershed and sewerage system under consideration must be constant during the data collecting period and must remain constant for the period of time in the future for which projections are to be made. This condition is very difficult to be met in watersheds that are becoming increasingly more urbanized, but it may be assumed to be met for older, well developed watersheds. However, if the manner in which the changing characteristics affecting the runoff from the watershed and the flow through the sewerage system and ultimately the overflows can be determined, the distributions may be adjusted accordingly. The second condition is that the data collecting period must extend over a period of many years which include both extremes of precipitation and runoff.

The data required to be collected in using this method are the volume of each individual overflow which occurs at the outlet and the time between each overflow. The volumes may be found by measuring the flow rates of the overflows with respect to time from which the overflow hydrographs may be drawn. If it is not convenient to measure the actual overflows, the sewer discharge versus time may be measured near the outlet. From the sewer discharge hydrograph, the portion of the flow which is intercepted is subtracted which gives the overflow hydrograph. The volume of overflow is found by finding the area under the overflow hydrograph.

The dates and the times at the beginning and end of each overflow should be recorded. It is best to calculate the time between overflows by both the methods discussed previously; that is, from the end of one overflow to the end of the succeeding overflow and from the end of one overflow to the beginning of the next overflow. This should be done since it may be necessary to consider both uncontrolled and controlled storage outlets. Data concerning the precipitation causing overflows such as date, time, depth, and duration should also be recorded.

Tabulation of required overflow data for the Bloody Run Sewer Watershed is presented in Table 10-1

GENERATION OF MISSING DATA AND GENERALIZATION OF OBSERVED DATA

Due to malfunction of equipment or other reasons, it is possible that data for some overflows during the sampling period may be missing. These missing data may be generated to varying degrees of accuracy using a number of methods. Among the methods that may be used are the regression analysis method (where a regression equation for overflow is found from previous observations using as variables, precipitation depth, duration and antecedent conditions), one of the various hydrograph methods or a deterministic mathematical model.

All of these methods require a knowledge of the precipitation causing the overflow which is missing from the set of data. It is best to obtain the precipitation data from raingages located within the watershed under consideration; however, National Weather Service precipitation data from a nearby station may be used if necessary.

The regression analysis method is probably the least accurate but the simplest to use. The deterministic mathematical models, such as the EPA Storm Water Management Model [24] and the UCUR Model [23] are very accurate; however, they require a considerable amount of effort to

Table 10-1. PRECIPITATION AND OVERFLOW DATA.
AUGUST 19, 1970 TO JULY 14, 1971.
BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO

Date	Average Precipitation Depth Inches	Average Precipitation Duration Minutes	Overflow Volume Acre-Feet	Time Interval Between Overflows Hours
8/19/70	.17	30	11.260	-
8/20/70	.43	26	14.263	29
8/22/70	.79	65	21.648	43
9/3/70	.40	115	10.795	287
9/5/70	.07	45	3.698	29
9/8/70	.44	55	10.909	87
9/13/70	.99	31	13.027	124
9/21/70	.05	10	2.231	183
9/22/70	.49	20	7.200	29
9/24/70	.22	30	6.198	43
9/25/70	1.13	151	17.583	12
9/26/70	.40	80	9.783	27
9/26-27/70	.19	175	7.589	5
9/27/70	.12	240	0.684	3
10/9-10/70	.36	234	4.087	307
10/12/70	.30	803	41.322	66
10/13/70	.18	160	16.694	2
10/14/70	.44	300	44.735	24
10/20/70	.72	344	52.253	141
10/20/70	.06	105	5.519	2
10/20/70	.05	41	6.265	2
10/20/70	.16	202	13.596	1
10/29/70	.12	326	1.800	204

Table 10-1 (continued).

Date	Average Precipitation Depth Inches	Average Precipitation Duration Minutes	Overflow Volume Acre-Feet	Time Interval Between Overflows Hours
10/29/70	.05	31	0.076	3
10/29/70	.12	116	3.568	1
11/2/70	.22	66	11.153	85
11/2/70	.31	133	40.922	2
11/9/70	.11	78	6.665	165
11/14/70	.69	559	21.665	109
11/19/70	.10	32	2.844	125
11/20/70	.76	250	85.422	2
11/20/70	.08	29	2.088	3
12/3/70	.13	15	7.571	327
12/11/70	.08	15	5.065	184
12/11/70	.28	195	5.665	4
12/12/70	.54	365	73.203	5
12/16/70	.27	215	15.974	96
12/16/70	.30	214	40.856	1
12/16-17/70	.56	338	88.532	4
12/21/70	.38	286	32.147	103
12/21-22/70	.74	420	141.407	12
12/23/70	.34	198	51.253	20
1/3-4/71	-	-	100**	283
1/13/71	.31	213	30.103	220
1/13/71	.26	223	24.127	1
1/14/71	.34	145	53.119	7

Table 10-1 (continued).

Date	Average Precipitation Depth Inches	Average Precipitation Duration Minutes	Overflow Volume Acre-Feet	Time Interval Between Overflows Hours
2/4/71	.96	485	173.154	502
2/4-5/71	.15	128	17.595	7
2/5/71	.17	21	29.059	2
2/11/71	*	*	0.435	164
2/12/71	*	*	13.276	14
2/12/71	*	*	37.146	1
2/12-13/71	*	*	11.677	1
2/17/71	.13	81	48.911	102
2/19/71	.25	140	37.057	51
2/21-22-23/71	2.47	1149	667.368	47
2/26/71	.06	65	0.227	78
2/26/71	.07	98	2.870	1
2/26/71	.06	38	6.914	8
3/6/71	.61	468	104.132	200
3/10/71	*	*	83.196	58
3/19/71	*	*	21.083	203
4/1/71	.13	148	0.196	320
4/1/71	.42	323	48.307	2
4/6/71	.12	68	0.938	126
4/13/71	.25	278	23.318	181
4/27/71	.46	68	28.526	94
5/2/71	.06	80	0.058	122
5/2/71	.09	90	2.590	1

Table 10-1 (continued).

Date	Average Precipitation Depth Inches	Average Precipitation Duration Minutes	Overflow Volume Acre-Feet	Time Interval Between Overflows Hours
5/5/71	.18	55	15.623	88
5/6/71	.09	95	1.555	6
5/6/71	.39	103	53.364	1
5/6/71	.07	45	1.466	5
5/7/71	1.22	530	202.879	22
5/13/71	.28	162	21.621	126
5/13/71	.14	254	1.217	4
5/19/71	.15	33	12.334	154
5/24-25/71	.75	280	98.729	122
6/7/71	.22	35	1.964	293
6/7/71	.13	35	6.132	7
6/14/71	.87	84	58.296	167
6/15/71	.25	148	21.630	17
6/19/71	.36	240	27.779	97
6/28/71	1.05	119	68.737	213
7/10/71	1.66	70	140.140	286
7/11/71	.15	128	8.247	8
7/11/71	.17	110	25.966	10
7/14/71	.24	28	26.837	58

*Overflow caused by snow fall or snow melt.
**Overflow volume estimated from National Weather Service precipitation data.

obtain and prepare detailed information on the characteristics of the watershed.

It is assumed herein that during planning and design of a storage project, sufficient time will be available to obtain data on overflows for a period of approximately one year. To assume that the sampling period would extend for a longer period is unrealistic, however, the sampling period should be as long as possible. Since data can be gathered over only a relatively short period, it is necessary to generalize the data to represent long term overflow characteristics.

A method to generalize short term data is used to show the approximate adjustment needed in the overflow volume distribution. Hourly precipitation data for a nearby National Weather Service gaging station covering a period of time sufficiently long, about 10 years, should be obtained. The monthly precipitation depths and the number of times precipitation occurred, which based on judgment would result in individual overflow events, can be obtained from these records. It is granted that the determination of the precipitation causing individual overflows is somewhat arbitrary, but a close approximation should be obtainable. The average monthly depths and the estimate of average monthly overflows may be compared to the actual short term data to observe variations. Also, from these records the depths and durations of all extreme precipitation, say greater than 2 inches, which result in individual overflow events are obtained. Overflow volumes for the extreme precipitation events are found by using one of the methods mentioned previously. These overflow volumes are arranged in increasing order of magnitude. Using the estimated number of overflows, N, for the period of record and the plotting position, $M/(N + 1)$, where $M = N - n_o + 1$ to N and n_o = the number of overflows caused by extreme precipitation, the overflow volumes are plotted on the probability paper used for the original measured overflow volumes. A new straight line may then be drawn including the extreme overflow volumes. This new line represents the generalized overflow volume distribution.

CASE STUDY

The Bloody Run Combined Sewer System in Cincinnati, Ohio has been selected to demonstrate the application of the developed method. The Bloody Run Watershed is described in Section 3 and 4 of Volume II of this report. Included in the data gathered from the Bloody Run Watershed by the

University of Cincinnati are measurements of flow at the outlet during storm runoff periods and recordings of precipitation. The precipitation is measured by three recording type raingages. Data are available corresponding to every storm which produced significant runoff since the middle of August, 1970. The data used in this case study are from storms occurring from mid August, 1970 to mid July, 1971.

At the outlet, an interceptor sewer transports the dry weather flow to the waste treatment plant. During runoff periods much of the flow overflows and untreated enters the Mill Creek Stream. The amount of flow that is allowed to enter the interceptor is controlled by a float operated gate.

During 1952, the Department of Sewers of the City of Cincinnati estimated the average dry weather flow at the outlet at 6.81 mgd. The regulating mechanism at the inlet to the interceptor has been set to allow a maximum flow into the interceptor of three times the average dry weather flow, i.e., 20.43 mgd or 31.6 cfs.

In order to determine the volume of each overflow event, the hydrographs of the flow at the outlet during periods of overflow were drawn. The amount of flow intercepted was subtracted from the hydrographs giving the overflow hydrographs. The areas under the overflow hydrographs were found with the use of a planimeter giving the overflow volumes.

In determining the time intervals between overflows, the assumption was made that during overflow periods no outflow to the interceptor could be allowed from a storage facility since the interceptor would be flowing to capacity. As was described earlier, the time during which outflow was possible determined the time interval between overflows. Since outflow was not possible during overflows, the time between overflows was measured from the end of one overflow event to the end of the next overflow event. This time interval was found to the nearest whole hour. If the time interval between two overflows was less than one half hour, the two overflows were considered as one overflow and their volumes were added. This was a result of measuring the time interval to the nearest whole hour.

The average depths and durations of precipitation for the watershed which caused the overflows were found. The depths and durations of precipitation for the three raingages were tabulated and a weighted average determined according to the Thiessen method [28].

In Table 10-1 a tabulation of the dates of occurrence, average depths of precipitation, durations of precipitation, overflow volumes, and time intervals between overflows is given.

The overflow volumes were then ordered from smallest to largest and the plotting position of each was calculated using m/(n + 1) where n is the total number of overflow events and m is the order number from 1 to n. In Table 10-2 the ordered overflows and their respective plotting positions have been tabulated. The overflow volumes were then plotted on lognormal probability paper. The plotted points closely approximated a straight line as shown in Figure 10-2 which indicated that the lognormal distribution was a good model of the observed data.

A straight line was fit to the plotted points. This line represented the theoretical distribution by which the observed distribution was approximated. The mean, μ, and the standard deviation, σ, of the theoretical distribution was then found. Chow [26] has given the equations for the mean and standard deviation of the lognormal distribution as follows:

$$\mu = \exp\left(\mu_y + \frac{\sigma_y^2}{2}\right)$$

$$\sigma = \mu\left(\exp \sigma_y^2 - 1\right)^{1/2} \qquad (5)$$

where, $y = \ln x$.

Since in the lognormal distribution the logarithm of the variate is normally distributed, the mean, μ_y, and the mean plus one standard deviation, $\mu_y + \sigma_y$, of the logarithm of the variate is at 50 percent and 84.13 percent respectively. Using these relations, the values of the mean and standard deviation for the overflow volume distribution were found to be 42.9 and 135.9 respectively.

The required storage volumes were calculated next. The value of the storage outflow rate, O, was taken as the difference between the interceptor capacity and the average dry weather flow. This value was found to be 1.74 acre-feet per hour. The required storage volumes were calculated using equation (4). Table 10-3 gives the tabulation of the dates of occurrence and the required storage volumes which were arranged in order of increasing magnitude and their plotting positions calculated as shown in Table 10-4. The

Table 10-2. OVERFLOW VOLUMES AND PLOTTING POSITIONS ARRANGED IN ORDER OF DECREASING MAGNITUDE FROM AUGUST 19, 1970 TO JULY 14, 1971. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Order No. m	Date	Overflow Volume Acre-Feet	Plotting Position $\frac{m}{n+1}$
1	5/2/71	.058	.01124
2	10/29/70	.076	.02247
3	4/1/71	.196	.03371
4	2/26/71	.227	.04494
5	2/11/71	.435	.05618
6	9/27/70	.684	.06742
7	4/6/71	.938	.07865
8	5/13/71	1.217	.08989
9	5/6/71	1.466	.1011
10	5/6/71	1.555	.1124
11	10/29/70	1.800	.1236
12	6/7/71	1.964	.1348
13	11/20/70	2.088	.1461
14	9/21/70	2.231	.1573
15	5/2/71	2.590	.1685
16	11/19/70	2.844	.1798
17	2/26/71	2.870	.1910
18	10/29/70	3.568	.2022
19	9/5/70	3.698	.2135
20	10/9/70	4.087	.2247
21	12/11/70	5.065	.2360
22	10/20/70	5.519	.2472
23	12/11/70	5.665	.2584
24	6/7/71	6.132	.2697
25	9/24/70	6.198	.2809
26	10/20/70	6.265	.2921
27	11/9/70	6.665	.3034
28	2/26/71	6.914	.3146
29	9/22/70	7.200	.3258
30	12/3/70	7.571	.3371
31	9/26-27/70	7.589	.3483
32	7/11/71	8.247	.3596
33	9/26/70	9.783	.3708
34	9/3/70	10.795	.3820
35	9/8/70	10.909	.3933
36	11/2/70	11.153	.4045
37	8/19/70	11.260	.4157

Table 10-2 (continued).

Order No. m	Date	Overflow Volume Acre-Feet	Plotting Position $\frac{m}{n+1}$
38	2/12-13/71	11.677	.4270
39	5/19/71	12.334	.4382
40	9/13/70	13.027	.4494
41	2/12/71	13.276	.4607
42	10/20/70	13.596	.4719
43	8/20/70	14.263	.4831
44	5/5/71	15.623	.4944
45	12/16/70	15.974	.5056
46	10/13/70	16.694	.5169
47	9/25/70	17.58[illegible]	.5281
48	2/4-5/71	17.595	.5393
49	3/19/71	21.083	.5506
50	5/13/71	21.621	.5618
51	6/15/71	21.630	.5730
52	8/22/70	21.648	.5843
53	11/14/70	21.665	.5955
54	4/13/71	23.318	.6067
55	1/13/71	24.127	.6180
56	7/11/71	25.966	.6292
57	7/14/71	26.837	.6404
58	6/19/71	27.779	.6517
59	4/27/71	28.526	.6629
60	2/5/71	29.059	.6966
61	1/13/71	30.103	.6854
62	12/21/70	32.147	.6966
63	2/19/71	37.057	.7079
64	2/12/71	37.146	.7191
65	12/16/70	40.856	.7303
66	11/2/70	40.922	.7416
67	10/12/70	41.322	.7528
68	10/14/70	44.735	.7640
69	4/1/71	48.503	.7753
70	2/17/71	48.911	.7865
71	12/23/70	51.253	.7978
72	10/20/70	52.253	.8090

Table 10-2 (continued).

Order No. m	Date	Overflow Volume Acre-Feet	Plotting Position $\frac{m}{n+1}$
73	1/14/71	53.119	.8202
74	5/6/71	53.364	.8315
75	6/14/71	58.296	.8427
76	6/28/71	68.737	.8539
77	12/12/70	73.203	.8652
78	3/10/71	83.196	.8764
79	11/20/70	85.422	.8876
80	12/16-17/70	88.532	.8989
81	5/24-25/71	98.729	.9101
82	1/3-4/71	100	.9213
83	3/6/71	104.132	.9326
84	7/10/71	140.140	.9438
85	12/21-22/70	141.407	.95506
86	2/4/71	173.154	.96629
87	5/7/71	202.879	.97753
88	2/21-22-23/71	667.368	.98876

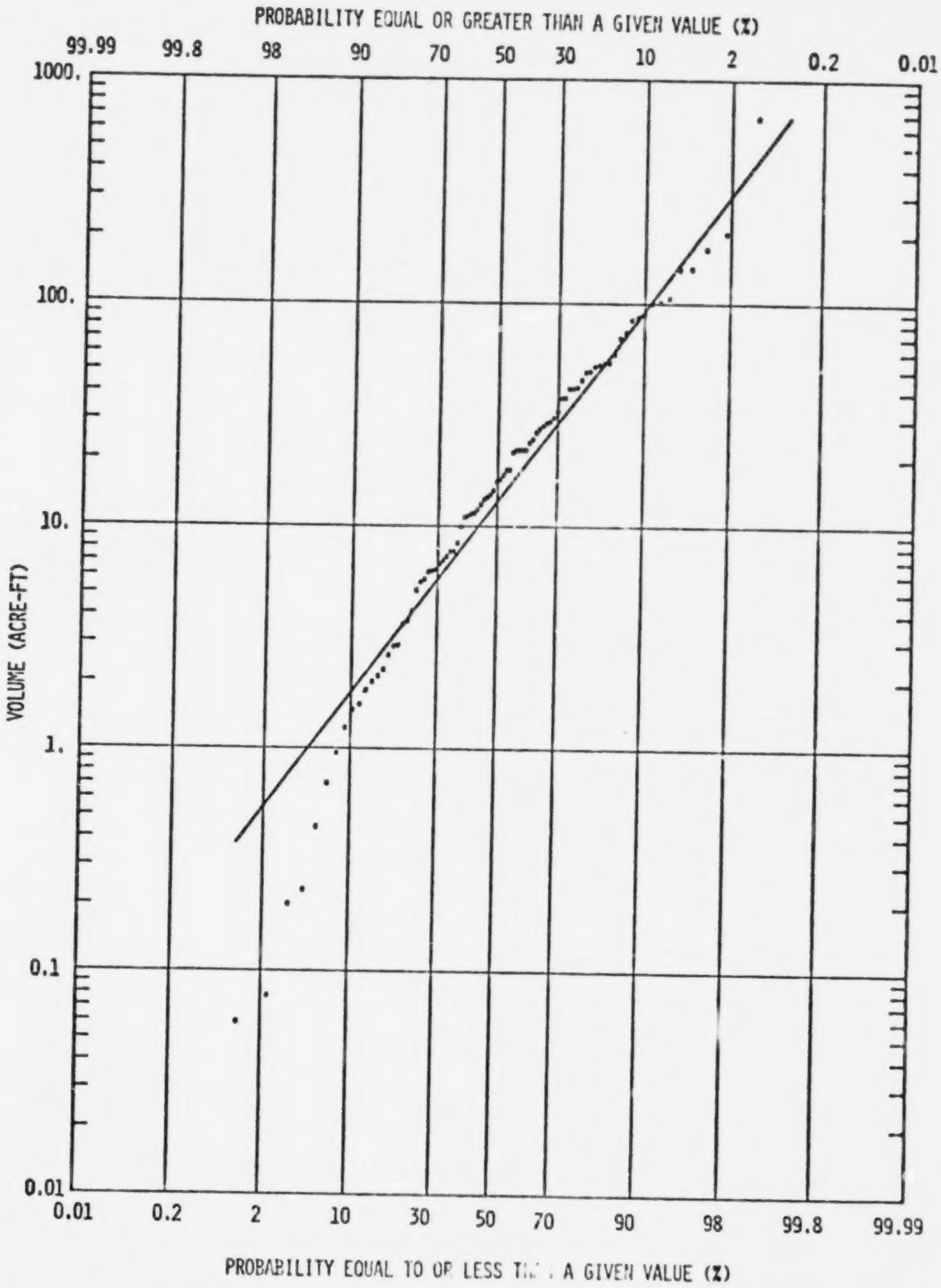


Figure 10-2. CUMULATIVE DISTRIBUTION FUNCTION OF OVERFLOW VOLUMES. BLOODY RUN SEWER WATERSHED,CINCINNATI,OHIO.

Table 10-3. REQUIRED STORAGE VOLUMES FOR DIFFERENT STORAGE OUTFLOW CAPACITIES FROM AUGUST 19, 1970 TO JULY 14, 1971. BLOODY RUN WATERSHED, CINCINNATI, OHIO.

Date	Time Interval Between Overflows Hours	Overflow Volumes Acre-Ft.	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet			
			O=.5	O=1.	O=1.74	O=10.
8/19/70	-	11.260	-	-	-	-
8/20/70	29	14.263	14.263	14.263	14.263	14.263
8/22/70	43	21.648	21.648	21.648	21.648	21.648
9/3/70	287	10.795	10.795	10.795	10.795	10.795
9/5/70	29	3.698	3.698	3.698	3.698	3.698
9/8/70	87	10.909	10.909	10.909	10.909	10.909
9/13/70	124	13.027	13.027	13.027	13.027	13.027
9/21/70	183	2.231	2.231	2.231	2.231	2.231
9/22/70	29	7.200	7.200	7.200	7.200	7.200
9/24/70	43	6.198	6.198	6.198	6.198	6.198
9/25/70	12	17.583	17.698	17.500	17.500	17.500
9/26/70	27	9.783	13.981	9.783	9.783	9.783
9/26-27/70	5	7.589	19.070	12.372	8.694	7.589
9/27/70	3	0.684	18.254	10.056	4.172	0.684
10/9-10/70	307	4.087	4.087	4.087	4.087	4.087
10/12/70	66	41.322	41.322	41.322	41.322	41.322
10/13/70	2	16.694	57.016	56.016	54.545	38.016
10/14/70	24	44.735	89.751	76.751	57.627	44.735
10/20/70	141	52.253	71.504	52.253	52.253	52.253
10/20/70	2	5.519	76.023	55.772	54.301	37.772
10/20/70	2	6.265	81.288	60.037	57.095	24.037
10/20/70	1	13.596	94.384	72.633	68.955	27.633
10/29/70	204	1.800	1.800	1.800	1.800	1.800

Table 10-3. (continued).

Date	Time Interval Between Over-flows Hours	Overflow Volumes Acre-Ft.	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet			
			O=.5	O=1.	O=1.74	O=10.
10/29/70	3	0.076	0.376	0.076	0.076	0.076
10/29/70	1	3.568	3.568	3.568	3.568	3.568
11/2/70	85	11.153	11.153	11.153	11.153	11.153
11/2/70	2	40.922	51.075	50.075	48.604	40.922
11/9/70	165	6.665	6.665	6.665	6.665	6.665
11/14/70	109	21.665	21.665	21.665	21.665	21.665
11/19/70	125	2.844	2.844	2.844	2.844	2.844
11/20/70	2	85.422	87.266	86.266	85.422	85.422
11/20/70	3	2.088	87.854	85.354	82.303	57.510
12/3/70	327	7.571	7.571	7.571	7.571	7.571
12/11/70	184	5.065	5.065	5.065	5.065	5.065
12/11/70	4	5.665	8.730	6.730	5.665	5.665
12/12/70	5	73.203	79.433	74.933	73.203	73.203
12/16/70	96	15.974	47.407	15.974	15.974	15.974
12/16/70	1	40.856	87.763	55.830	55.094	46.830
12/16-17/70	4	88.532	174.295	140.362	136.684	95.362
12/21/70	103	32.147	154.942	69.509	32.147	32.147
12/21-22/70	12	141.407	290.349	198.916	152.728	141.407
12/23/70	20	51.253	331.602	230.169	169.270	51.253
1/3-4/71	283	100.000	290.102	100.000	100.000	100.000
1/13/71	220	30.103	210.205	30.103	30.103	30.103
1/13/71	1	24.127	233.832	53.230	52.494	44.230
1/14/71	7	53.119	283.451	99.349	93.465	53.119

Table 10-3. (continued).

Date	Time Interval Between Over-flows Hours	Overflow Volumes Acre-Ft.	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet			
			O=.5	O=1.	O=1.74	O=10.
2/4/71	502	173.154	205.605	173.154	173.154	173.154
2/4-5/71	7	17.595	219.700	183.749	178.600	120.749
2/5/71	2	29.059	247.759	210.808	204.188	129.808
2/11/71	164	0.435	166.194	47.243	0.435	0.435
2/12/71	14	13.276	172.470	46.519	13.276	13.276
2/12/71	1	37.146	209.116	82.665	48.686	40.422
2/12-13/71	1	11.677	220.293	93.342	58.628	42.099
2/17/71	102	48.911	218.204	48.911	48.911	48.911
2/19/71	51	37.057	229.761	37.057	37.057	37.057
2/21-22-23/71	47	667.368	873.629	667.368	667.368	667.368
2/26/71	78	0.227	834.856	589.595	532.223	0.227
2/26/71	1	2.870	837.226	591.465	533.357	2.870
2/26/71	8	6.914	840.140	590.379	526.387	6.914
3/6/71	200	104.132	844.271	494.511	283.412	104.132
3/10/71	58	83.196	898.467	519.707	265.947	83.196
3/19/71	203	21.083	818.050	337.790	21.083	21.083
4/1/71	320	0.196	658.246	17.986	0.196	0.196
4/1/71	2	48.307	705.553	64.293	48.307	48.307
4/6/71	126	0.938	643.491	0.938	0.938	0.938
4/13/71	181	23.318	576.309	23.318	23.318	23.318
4/27/71	94	28.526	557.835	28.526	28.526	28.526
5/2/71	122	0.058	496.893	0.058	0.058	0.058
5/2/71	1	2.590	498.983	2.590	2.590	2.590

Table 10-3. (continued).

Date	Time Interval Between Overflows Hours	Overflow Volumes Acre-Ft.	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet			
			O = .5	O=1.	O=1.74	O=10.
5/5/71	88	15.623	470.606	15.623	15.623	15.623
5/6/71	6	1.555	469.161	11.178	6.765	1.555
5/6/71	1	53.364	522.025	63.542	58.393	53.364
5/6/71	5	1.466	520.991	60.008	51.182	4.830
5/7/71	22	202.879	712.870	240.887	215.879	202.879
5/13/71	126	21.621	671.491	136.508	21.621	21.621
5/13/71	4	1.217	670.708	133.725	15.896	1.217
5/19/71	154	12.334	606.042	12.334	12.334	12.334
5/24-25/71	122	98.729	643.770	98.729	98.729	98.729
6/7/71	293	1.964	499.234	1.964	1.964	1.964
6/7/71	7	6.132	501.866	6.132	6.132	6.132
6/14/71	167	58.296	476.662	58.296	58.296	58.296
6/15/71	17	21.630	489.792	62.926	50.422	21.630
6/19/71	97	27.779	469.071	27.779	27.779	27.779
6/28/71	213	68.737	431.308	68.737	68.737	68.737
7/10/71	286	140.140	428.448	140.140	140.140	140.140
7/11/71	8	8.247	432.695	140.387	134.503	68.387
7/11/71	10	25.966	453.661	156.353	143.113	25.966
7/14/71	58	26.837	451.498	125.190	69.289	26.837

Table 10-4. REQUIRED STORAGE VOLUMES AND PLOTTING POSITIONS ARRANGED IN ORDER OF INCREASING MAGNITUDE FROM AUGUST 19, 1970 TO JULY 14, 1971. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Order No. m	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, In Acre-Feet/Hour Acre-Feet				Plotting Position $\frac{m}{n+1}$
	O=.5	O=1.	O=1.74	O=10.	
1	0.375	0.058	0.058	0.058	.01136
2	1.800	0.076	0.076	0.076	.02273
3	2.231	0.938	0.196	0.196	.03409
4	2.844	1.800	0.435	0.227	.04545
5	3.568	1.964	0.938	0.435	.0568
6	3.698	2.231	1.800	0.684	.0682
7	4.087	2.590	1.964	0.938	.0795
8	5.065	2.844	2.231	1.217	.0909
9	6.198	3.568	2.590	1.555	.1023
10	6.665	3.698	2.844	1.800	.1136
11	7.200	4.087	3.568	1.964	.1250
12	7.571	5.065	3.698	2.231	.1364
13	8.730	6.132	4.087	2.590	.1477
14	10.795	6.198	4.171	2.844	.1591
15	10.909	6.665	5.065	2.870	.1705
16	11.153	6.730	5.665	3.568	.1818
17	13.027	7.200	6.132	3.698	.1932
18	13.980	7.571	6.198	4.087	.2045
19	14.263	9.783	6.665	4.829	.2159
20	17.698	10.055	6.764	5.065	.2273
21	18.253	10.795	7.200	5.665	.2386
22	19.069	10.909	7.571	6.132	.2500
23	21.648	11.153	8.694	6.198	.2614
24	21.665	11.177	9.783	6.665	.2727

Table 10-4.(continued).

Order No. m	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet				Plotting Position $\frac{m}{n+1}$
	O=.5	O=1.	O=1.74	O=10.	
25	41.322	12.335	10.795	6.914	.2841
26	47.406	12.371	10.909	7.200	.2955
27	51.075	13.027	11.153	7.571	.3068
28	57.015	14.263	12.334	7.589	.3182
29	71.503	15.623	13.027	9.783	.3295
30	76.022	15.974	13.276	10.795	.3409
31	79.432	17.500	14.263	10.909	.3523
32	81.287	17.985	15.623	11.153	.3636
33	87.266	21.648	15.895	12.334	.3750
34	87.762	21.665	15.974	13.027	.3864
35	87.854	23.318	17.500	13.276	.3977
36	89.750	27.779	21.083	14.263	.4091
37	94.383	28.526	21.621	15.623	.4205
38	154.941	30.103	21.648	15.974	.4318
39	166.193	37.057	21.665	17.500	.4432
40	172.469	41.322	23.318	21.083	.4545
41	174.294	46.518	27.779	21.621	.4659
42	205.604	47.242	28.526	21.630	.4773
43	209.115	48.911	30.103	21.648	.4886
44	210.204	50.075	32.147	21.665	.5000
45	218.203	52.253	37.057	23.318	.5114
46	219.699	53.230	41.322	24.036	.5227
47	220.292	55.772	48.307	25.966	.5341
48	229.760	55.830	48.603	26.837	.5455

Table 10-4. (continued).

Order No. m	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet				Plotting Position $\frac{m}{n+1}$
	O=.5	O=1.	O=1.74	O=10.	
49	233.831	56.015	48.686	27.632	.5568
50	247.758	58.296	48.911	27.779	.5682
51	283.450	60.007	50.421	28.526	.5795
52	290.101	60.037	51.181	30.103	.5909
53	290.348	62.926	52.253	32.147	.6023
54	331.601	63.541	52.494	37.057	.6136
55	428.448	64.292	54.300	37.772	.6250
56	431.308	68.737	54.544	38.015	.6364
57	432.695	69.508	55.094	40.421	.6477
58	451.497	72.632	57.094	40.922	.6591
59	453.661	74.932	57.627	41.322	.6705
60	469.071	76.750	58.296	42.098	.6818
61	469.160	82.664	58.393	44.230	.6932
62	470.605	85.354	58.627	44.735	.7045
63	476.662	86.266	68.737	46.830	.7159
64	489.792	93.341	68.955	48.307	.7273
65	496.892	98.729	69.289	48.911	.7386
66	498.982	99.348	73.203	51.253	.7500
67	499.234	100.000	82.303	52.253	.7614
68	501.866	125.189	85.422	53.119	.7727
69	520.990	133.724	93.464	53.364	.7841
70	522.024	136.507	98.729	57.510	.7955
71	557.834	140.140	100.000	58.296	.8068
72	576.309	140.361	134.502	68.386	.8182

Table 10-4. (continued).

Order No. m	Required Storage Volumes, V', for Different Storage Outlet Capacities, O, in Acre-Feet/Hour Acre-Feet				Plotting Position $\frac{m}{n+1}$
	O=.5	O=1.	O=1.74	O=10.	
73	606.041	110.386	136.684	68.737	.8295
74	643.491	156.352	140.140	73.203	.8409
75	643.770	173.154	143.113	83.196	.8523
76	658.246	183.748	152.727	85.422	.8636
77	670.707	198.915	169.269	95.361	.8750
78	671.490	210.807	173.154	98.729	.8864
79	705.553	230.168	178.600	100.000	.8977
80	712.869	240.886	204.188	104.132	.9091
81	818.050	337.789	215.878	120.748	.9205
82	834.855	494.510	265.946	129.807	.9318
83	837.225	519.706	283.411	140.140	.9432
84	840.139	589.594	526.387	141.407	.95455
85	844.271	590.378	532.223	173.154	.96591
86	873.628	591.464	533.357	202.879	.97727
87	898.467	667.368	667.368	667.368	.98864

required storage volumes were plotted on lognormal probability paper and a straight line was fit as shown in Figure 10-3.

In order to observe the change in the distributions with a change in the storage outflow rate, O, the required storage volumes were calculated for O equal to .5, 1., and 10. acre-feet per hour. These required storage volumes have been tabulated in chronological order in Table 10-3 and in order of increasing magnitude in Table 10-4. Figure 10-4 shows the plot for the four required storage volume distributions corresponding to the different values of O.

In order to show the change in the mean and standard deviation for a change in O, the mean and standard deviation of the four distributions were found in the same manner that the mean and standard deviation of the overflow volumes were found and are given below:

for, $O = 0.5$, $\mu = 657.6$ and $\sigma = 4814.5$

$O = 1.0$, $\mu = 192.6$ and $\sigma = 919.0$

$O = 1.74$, $\mu = 120.2$ and $\sigma = 542.8$

$O = 10.0$, $\mu = 65.2$ and $\sigma = 241.7$.

Figure 10-5 is a graph of the mean and standard deviation of the required overflow volume distributions versus O.

From Figure 10-4 the required storage volume for each of the distributions was obtained such that at the end of any overflow event there was a probability of 2 percent that the storage volume would be equalled or exceeded. The values of 5400, 1500, 930, and 470 for 0 = .5, 1.0, 1.74, and 10 respectively were plotted versus O in Figure 10-6.

Since the observations of the overflows were taken over the very short period of 11 months, it was necessary to estimate how much the generalized distribution would differ from the observed distribution.

Precipitation data for the National Weather Service rain gage station at the Abbe Observatory in Cincinnati, near the Bloody Run Combined Sewer Watershed, was obtained for the 10 years including July 1961 through June 1971. The monthly precipitation depths were recorded along with an estimate of the number of storms causing overflows. It can be seen in Table 10-5 that, in general, the average monthly precipitation depths were somewhat greater than those recorded by the Weather Service during the period of overflow observation.

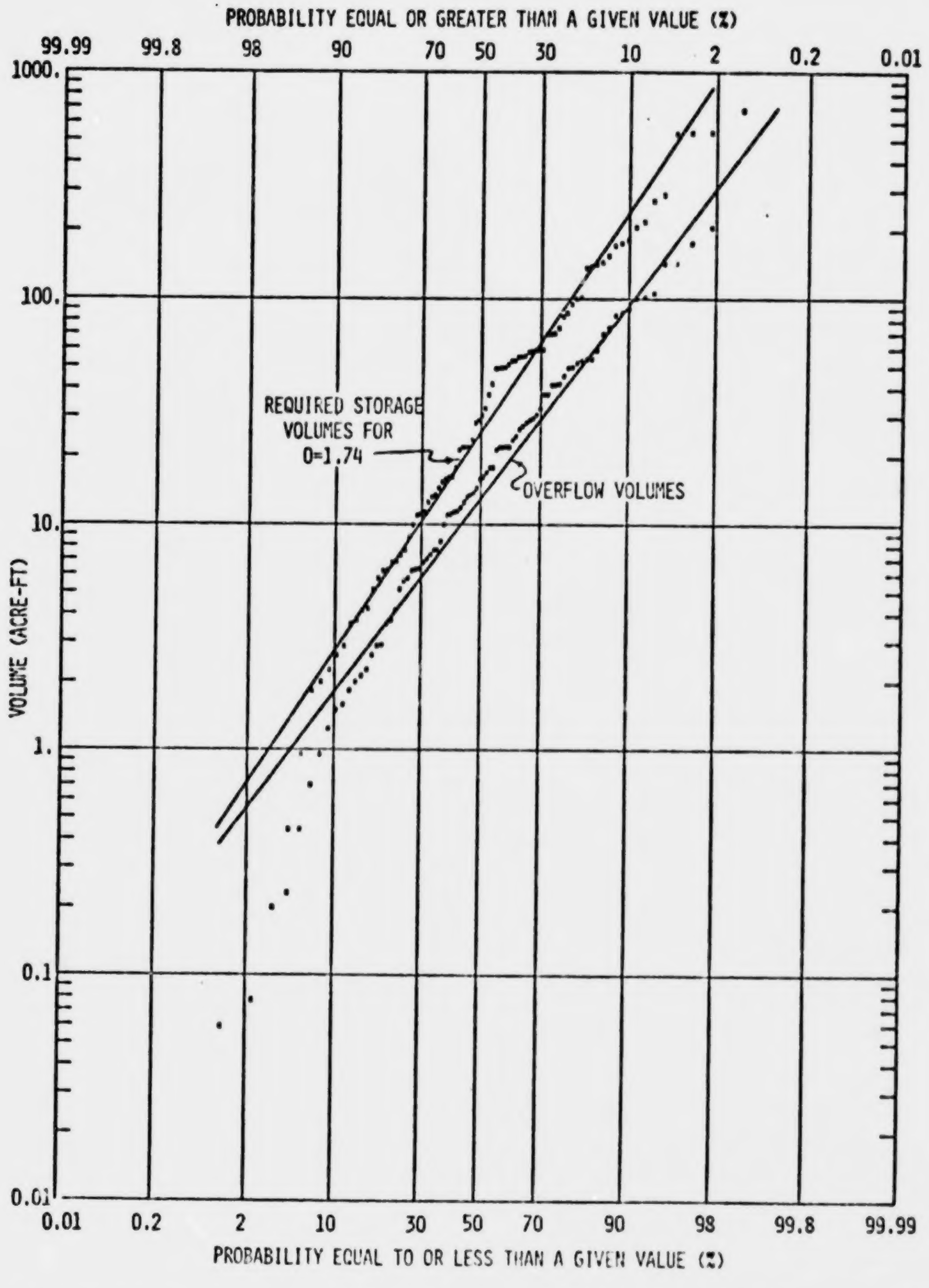


Figure 10-3. CUMULATIVE DISTRIBUTION FUNCTION OF OVERFLOW VOLUMES AND REQUIRED STORAGE VOLUMES FOR O = 1.74 IN ACRE-FEET/HOUR. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

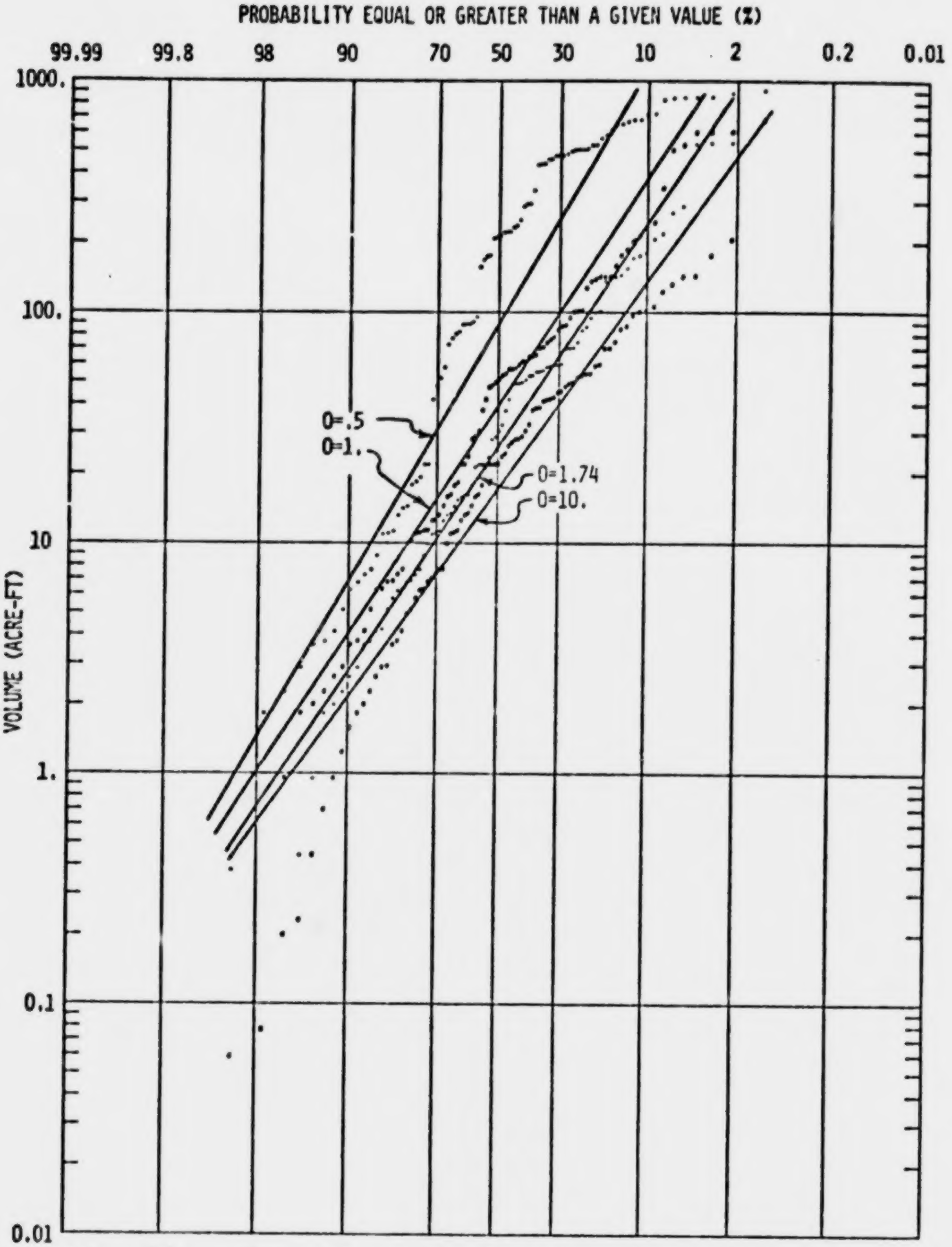


Figure 10-4. CUMULATIVE DISTRIBUTION FUNCTION OF REQUIRED STORAGE VOLUMES FOR DIFFERENT VALUES OF O IN ACRE-FEET/HOUR. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

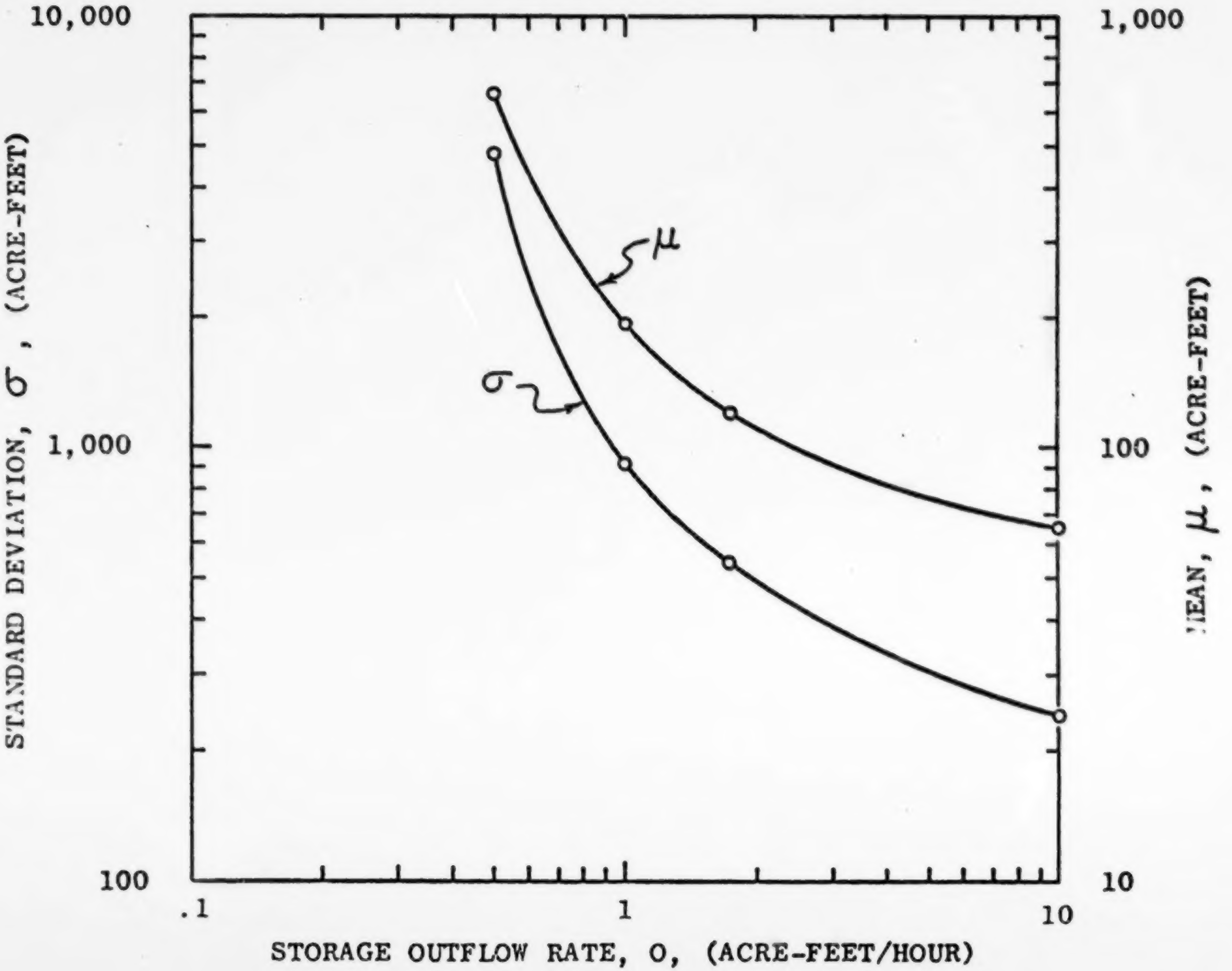


Figure 10-5. REQUIRED STORAGE VOLUME MEAN, μ, AND STANDARD DEVIATION σ, VERSUS STORAGE OUTFLOW RATE, O. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

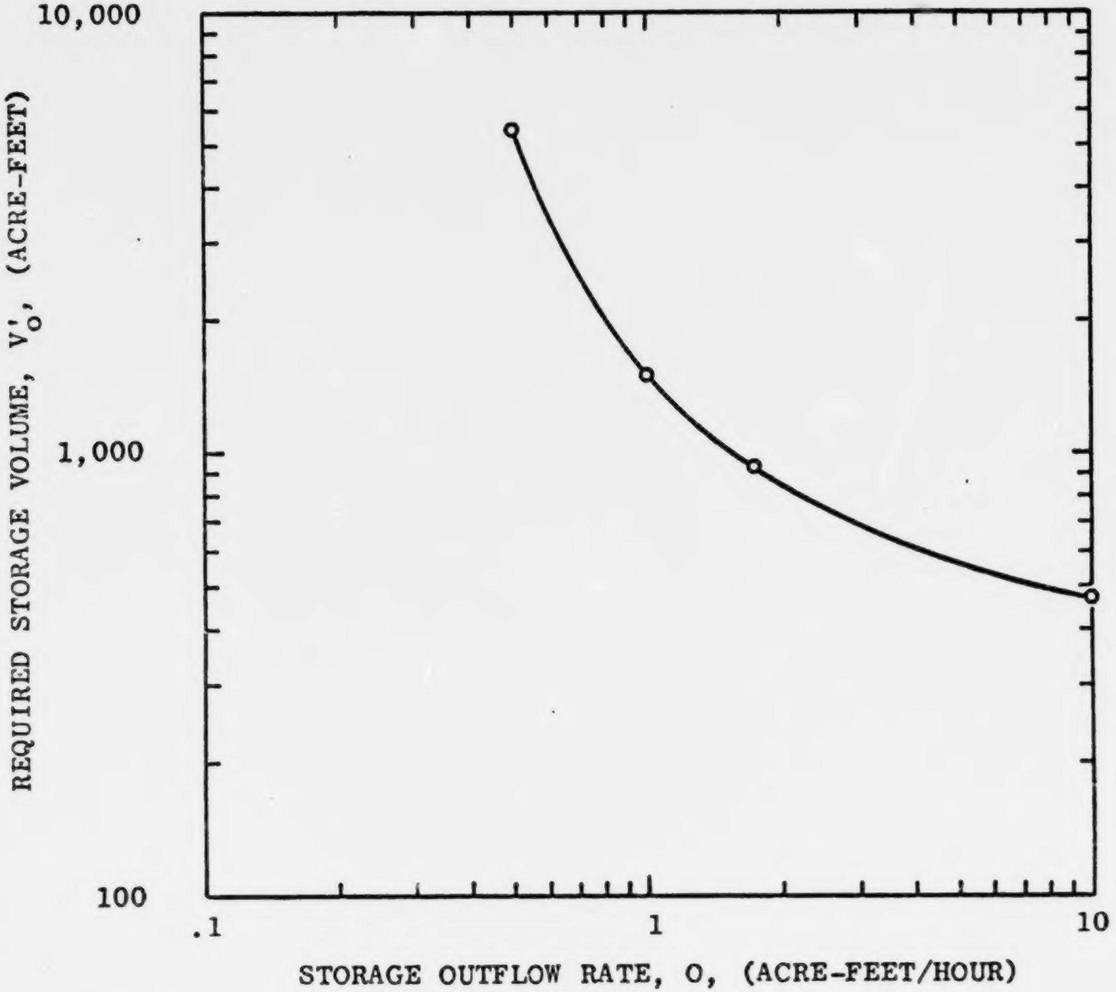


Figure 10-6. REQUIRED STORAGE VOLUME, V'_o, FOR A PROBABILITY, P $(V' \geq V'_o)$, EQUAL TO 2% VERSUS STORAGE OUTFLOW RATE, O. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Table 10-5. 10-YEAR AVERAGE MONTHLY PRECIPITATION DEPTHS AND THE MONTHLY PRECIPITATION DEPTHS FOR THE YEAR JULY, 1970 TO JUNE, 1971 FOR ABBE OBSERVATORY NEAR BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Month	10 Year Average Monthly Precipitation Depth. Inches	Monthly Precipitation Depths for Period of Observation. Inches
July	4.26	3.21
August	2.59	4.39
September	3.10	3.55
October	1.87	2.54
November	2.88	2.00
December	2.83	3.55
January	2.74	2.27
February	2.75	5.75
March	4.64	2.34
April	4.11	1.57
May	3.89	3.58
June	2.78	2.58

The estimate of the number of overflows for the 10 year period was 959.

The number of overflows estimated from the National Weather Service precipitation records from August 19, 1970 to June 28, 1971 was 80 compared to 84 observed during the same period. This discrepancy might have been due partially to errors in estimating and partially due to the fact that the Weather Service raingage did not measure the rain actually falling on the Bloody Run Watershed. However, the estimate of the number of overflows during the 10 year period was assumed to be sufficiently accurate.

From the observed Bloody Run overflow and precipitation data, a regression analysis was run. The model used for this regression analysis was:

$$V = b_o + b_1 x_1 + b_2 x_2 + b_3 x_3 + b_4 x_1^2 + b_5 x_2^2 + b_6 x_3^2 \quad (6)$$

where: V = observed overflow volume (acre-feet)

x_1 = observed precipitation depth (inches)

x_2 = observed precipitation duration (minutes)

x_3 = observed time interval since previous overflow (hours)

and b_o, b_1, b_2, b_3, b_4, b_5, b_6 are the regression coefficients to be found. The time interval since the previous overflow was used as a measure of antecedent conditions. The multiple correlation coefficient, using the three variables and their squares, was 0.9595. It was found that the time interval since the previous overflow and its square were the least significant in the regression equation. Dropping these two terms reduced the multiple correlation coefficient to 0.9559. The final regression equation found was,

$$V = -2.17 - 184.98\ x_1 + 485.20\ x_1^2 + 0.70\ x_2 - 0.00034\ x_2^2 \quad (7)$$

From the 10-year Weather Service records for Abbe Observatory, the depths and durations of all precipitation of 2 inches or greater, which appeared to have caused a single overflow, were estimated. The overflow volumes for these extreme precipitation depths were calculated using equation (7). The precipitation depths and durations and the calculated overflow volumes are given in Table 10-6. One overflow event was calculated by equation (7) and also was measured during the sampling period. The measured value of the overflow was 667.4 acre-feet and the calculated value was 634.6 acre-feet.

The calculated overflow volumes were ordered from lowest to largest. The lowest value was given the order number 946 and the largest 959. The plotting positions were obtained by $M/(N + 1)$ where $N = 959$. These values were plotted on lognormal probability paper along with the observed overflow volumes. This plot is shown in Figure 10-7. It is readily seen that the calculated extreme overflow volumes tend to emphasize the trend of the measured overflow volumes toward a straight line. If actual measurements of overflows had been taken over the 10 year period, there would have been in the neighborhood of 959 points plotted in Figure 10-2 as opposed to the 88 points actually plotted. Although the plotting positions of the overflow volumes observed at Bloody Run might have been changed, had this greater amount of data been available and included, it is reasonable that only the points plotted at the extremes, both high and low, would have had significant variation.

From this, it was assumed that the points in the center portion of the distribution of the actual observations adequately represented the center portion of the generalized distribution. Using these center points and the 10 year extreme overflow volumes, the generalized cumulative distribution function was obtained as indicated in Figure 10-7.

CONCLUSIONS AND RECOMMENDATIONS

A method for the design of retarding basins has been developed and the procedure has been demonstrated in a case study. This method uses a stochastic approach to the solution of the problem of finding a reasonable and adequate value for the design storage volume. The method basically consists of the following six steps:

1. Gather overflow and precipitation data.
2. Fit overflow volumes to a theoretical distribution. (For the Bloody Run Sewer Watershed the lognormal distribution was found to be satisfactory).
3. Calculate one or more sets of required storage volumes for values of the storage outflow rate.

Table 10-6. PRECIPITATION DEPTHS GREATER THAN 2 INCHES AND CORRESPONDING CALCULATED OVERFLOW VOLUMES FOR THE PERIOD JULY, 1961 TO JUNE, 1971. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

Date	Precipitation Depth Inches	Precipitation Duration Hours	Overflow Volumes Acre-Feet
7/4-5/61	2.66	5	692.8
3/20-21/62	2.05	25	446.1
3/4-5/63	3.77	37	1394.8
3/16/63	2.03	14	452.3
3/4/64	3.60	19	1372.0
3/8-9-10/64	5.74	51	3186.8
6/18/64	2.22	9	518.2
1/1/65	2.28	15	563.2
9/11-12/65	3.74	19	1480.6
10/20-21/65	2.12	29	453.5
5/22-23/68	2.00	14	440.1
5/23-24/68	2.58	11	703.5
4/1-2/70	2.44	17	641.8
2/21-22/71	2.43	20	634.6

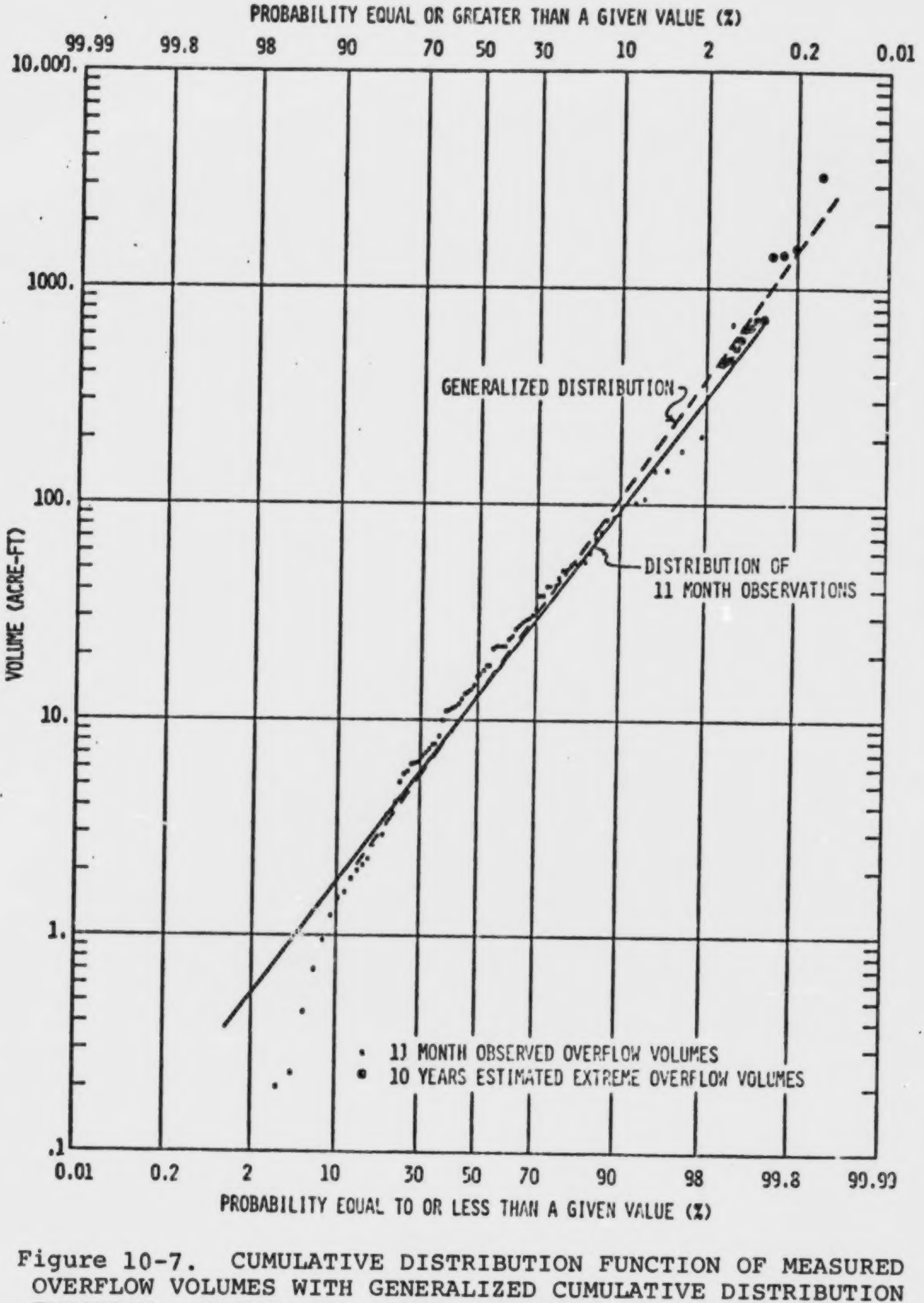


Figure 10-7. CUMULATIVE DISTRIBUTION FUNCTION OF MEASURED OVERFLOW VOLUMES WITH GENERALIZED CUMULATIVE DISTRIBUTION FUNCTION. BLOODY RUN SEWER WATERSHED, CINCINNATI, OHIO.

4. Fit required storage volumes to a theoretical distribution. (The lognormal distribution was found adequate for the Bloody Run Case Study).

5. Generalize the short term overflow distribution with the estimate of the long term overflow trend. In the Bloody Run Case Study, it was found that by including the calculated long term extreme overflow volumes in the distribution of the measured short term overflow volumes, the tendency of the overflow volumes to approximate a straight line on lognormal probability paper was increased.

Although it is believed that the basic concepts of this method, developed for the determination of the design capacity of retarding basins, are sound, additional work is recommended to verify the method for other combined sewerage systems and to broaden its scope to include storage at locations within a sewer system other than the outlet.

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SECTION 11

PUBLICATIONS

The following publications have been produced as a result of this project:

1. Preul, H.C., Papadakis, C.N., "Urban Runoff Characteristics", Research Report, Water Pollution Control Series, 11024 DQU 10/70, Water Quality Office, U.S. Environmental Protection Agency, Washington, D.C. October, 1970.

2. Papadakis, C.N., Preul, H.C., "University of Cincinnati Urban Runoff Model", Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY10, Proc. Paper 9298, Oct. 1972, pp. 1789-1804.

3. Preul, H.C., Papadakis, C.N., "Development of Design Storm Hyetographs for Cincinnati, Ohio, "Water Resources Bulletin, American Water Resources Association, Vol. 9, No. 2, April 1973, pp. 291-300.

4. Papadakis, C.N., Preul, H.C., "Infiltration and Antecedent Precipitation", Journal of the Hydraulics Dividion, ASCE, Vol. 99, No. HY8, Aug. 1973, pp. 1235-1245.

5. Papadakis, C.N., Preul, H.C., "Testing of Methods for Determination of Urban Runoff", Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY9, Sept. 1973, pp. 1319-1335.

6. Preul, H.C., "Assessment of Urban Runoff Quantity and Quality", Proceedings of the International Seminar and Exposition on Water Resources Instrumentation, Chicago, June, 1974; published by International Water Resources Association.

SECTION 12

GLOSSARY, ABBREVIATIONS AND SYMBOLS

Glossary

WATERSHED - The area which is drained by a river system.

DRAINAGE BASIN (STUDY AREA) - The area which contributes runoff to a stream at a given point (an individual section of a watershed).

SUBCATCHMENT - A subdivision of a drainage basin (generally determined by topography and pipe network configuration).

SUBAREA _ A subdivision of a subcatchment (generally based upon a single land use but may be identical to a subcatchment).

Abbreviations

ASCE	- American Society of Civil Engineers
EPA	- Environmental Protection Agency
SWMM	- Storm Water Management Model
UCURM	- University of Cincinnati Urban Runoff Model

BOD	- biochemical oxygen demand (5-day)
cfs	- cubic feet per second
COD	- chemical oxygen demand
DO	- dissolved oxygen
DWF	- dry weather flow
fpm	- feet per minute
fps	- feet per second
ft	- feet
gal.	- gallons
gal./capita/day	- gallons per capita per day
gpd	- gallons per day
gpm	- gallons per minute
gpm/sq ft	- gallons per minute per square foot
hr	- hour
in.	- inches
in./hr.	- inches per hour
lb.	- pounds
lb/acre/day	- pounds per acre per day
lb/acre/yr	- pounds per acre per year
lb/capita/day	- pounds per capita per day
lb/cf	- pounds per cubic foot

lb/day/cfs - pounds per day per cubic feet per second
lb/ft - pounds per foot
lb/sec - pounds per second
mgd - million gallons per day
min - minutes
mm - millimeters
ppm - parts per million
psf - pounds per square foot
psi - pounds per square inch
rpm - revolutions per minute
sec - second
sq ft - square feet
sq ft/min - square feet per minute
SS - suspended solids
tons/mo - tons per month
tons/sq mi/mo - tons per square mile per month
VSS - volatile suspended solids
yr - year

Symbols

Δ	delta
α	alpha
Σ	sigma
$<$	less than
$>$	greater than
∂	partial differentiation
ρ	rho
π	pi
θ	theta

SECTION 13

APPENDICES

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APPENDIX A

UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL

COMPUTER PROGRAM

```fortran
C     **********  ********************************************
C     INFILTRATION AND DEPRESSION STORAGE IN PERVIOUS AREAS
C     ****************************************************
C
C     DIVIDE THE TOTAL RAINFALL TIME IN N EQUAL SMALL INCREMENTS
C     READ N+1 VALUES OF AVAILABLE RAINFALL HYDROGRAPH(IN/HR)
C     EACH TIME INCREMENT DT IS IN MINUTES
C     AT TIME ZERO I=0. AT THE END OF THE TIME PERIOD I=N
C     RAINFALL INTENSITY AT TIME ZERO IS RO(FOR I=0)
C     RAINFALL INTENSITY AT A TIME T=DT*I IS RI(I)
C     AT A TIME T=DT*I,INFILTRATION CAPACITY IS F(I) AND ACTUAL
C     INFILTRATION IS AF(I) IN (IN/HR)
C     THE CONSTANT RATE AT WHICH F(I) APPROACHES ASYMPTOTICALLY
C     AS TIME CONTINUES,IS FC
C     THE RATE OF INFILTRATION CAPACITY WHEN TIME=0(I.E. I=0) IS FO
C     CK IS A POSSITIVE CONSTANT
C     DIFFERENCE BETWEEN RAINFALL AND ACTUAL INFILTRATION  S(I)
C     ACCUMULATED MASS PRECIPITATED UNTIL A TIME DT*I IS RM(I) INCHES
C     ACCUM.MASS ACTUALLY INFILTRATED UNTIL A TIME DT*I IS AFM(I) INCHES
C     WHERE RM(I) OR AFM(I) INDICATE THE MASS FROM TIME DT*(I-1) UNTIL
C     TIME DT*I ,PLUS THE PREVIOUS MASS
C     DIFFERENCE BETWEEN MASSES UNTIL A TIME DT*I IS P(I) IN INCHES
C     VOLUME OF WATER IN DEPRESSION STORAGE IS VS(I) INCHES
C     TOTAL DEPRESSION STORAGE CAPACITY SD INCHES
C     DEPRESSION STORAGE SUPPLY IS D(I)  INCH/HOUR
C     SS(I) IS THE RATE OF (RAIN-INFILTR-DEPRESSION STORAGE)
C     PP(I) IS THE ACCUMULATED MASS OF(RAIN-INFILTR-DEPRESSION STORAGE)
C     JPR=1 OBTAINS ALL PRINTOUT
C     JPR=N PRINTOUT EVERY N TIMES FROM SUBR.GUTEL AND PIROU
C     IPRINT=0 REDUCES UNNECESSARY PRINTOUT
      COMMON RI(120),AF(120),S(120),RM(120),AFM(120),P(120),D(120),
     1VS(120),SS(120),PP(120),SSIM(120),PPIM(120),VSIM(120),DIM(120),
     2DATE(8),DC(120),DE(120),OVQ(120,30),OVOW(120),NEW(30),QINL(120,40)
     3,QINLO(40),IW(600),V(600),N,DT,NN,NINL,QINP(600,40),QLEAV(600,40),
     4IUP(10),IDN(10),LNNEW(10),JPR,IPRINT
      WRITE(6,120)
  120 FORMAT('1')
      READ(5,21)(DATE(I),I=1,8),DT,N,IPRINT
   21 FORMAT(8A4,F5.2,2I5)
      READ (5,4) RO
    4 FORMAT(F5.2)
      READ(5,5) (RI(I),I=1,N)
    5 FORMAT (12F5.2)
      READ (5,6) FC,FO,CK,SD,SDIM
    6 FORMAT(3F10.5,2F8.4)
      READ(5,20)AN,JPR
   20 FORMAT(F5.2,I5)
      WRITE(6,501)
  501 FORMAT(5X,'*****************************************************
     1******',/,5X,'SIMULATION OF INFILTR.AND DEPRESSION STORAGE IN PERV
     2IOUS AREA',/,5X,'*************************************************
     3************',//)
      WRITE(6,22)(DATE(I),I=1,5)
   22 FORMAT(5X,'RAIN OF',6X,5A4,//)
      WRITE(6,23)(DATE(I),I=6,8)
   23 FORMAT(5X,'TIME OF RAIN START',3X,3A4,//)
      WRITE (6,605)
  605 FORMAT(11X,'FC',8X,'FO',9X,'CK')
      WRITE(6,606)FC,FO,CK
  606 FORMAT(10X,F4.2,6X,F4.2,6X,F6.4,//)
```

```
      WRITE(6,607) SD
  607 FORMAT(5X,'TOTAL DEPRESSION STORAGE CAPACITY OF THE',/,5X,'SUBCATC
     1HMENT IS    (IN INCHES)  =',15X,F8.4,//)
      IF (RO-FO) 10,7,7
    7 DO 8 I=1,N
    8 AF(I)=FC+(FO-FC)*EXP(-CK*DT*I)
      SO=RO-FO
      DO 9 I=1,N
    9 S(I)=RI(I)-AF(I)
   10 RM(1)=((RO+RI(1))/2.0)*DT/60.0
      IF (RO-FO) 19,12,12
   12 DO 11 I=2,N
   11 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 13 I=1,N
   13 AFM(I)=FC*DT*I/60.+(FO-FC)/(60.*CK)*(1.-EXP(-CK*DT*I))
      DO 14 I=1,N
   14 P(I)=RM(I)-AFM(I)
      OD=SO
      SSO=0.0
      NCON=0.0
      DO 800 I=1,N
      IF(RI(I)-AF(I)) 900,900,799
  900 D(I)=0.0
      NCON=1.0
      VS(I)=SD
      SS(I)=S(I)
      PP(I)=P(I)-SD
      GO TO 800
  799 IF(NCON) 798,798,900
  798 D(I)=S(I)*EXP(-P(I)/SD)
      VS(I)=SD*(1.0-EXP(-P(I)/SD))
      SS(I)=S(I)-D(I)
      PP(I)=P(I)-VS(I)
  800 CONTINUE
      WRITE (6,15)
   15 FORMAT(5X,'TIME FROM    RAINFALL        ACTUAL       DEPRESSION
     1OVERLAND ',/,5X,'RAIN START   INTENSITY   INFILTRATION   STOR.SUPP
     2LY    FLOWSUPPLY',/,5X,'(MINUTES)      (IN/HR)       (IN/HR)       (
     3IN/HR)        (IN/HR)  ',/)
      T=0.00
      WRITE(6,111) T,RO,FO,OD,SSO
  111 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      DO 17 I=1,N
      TT=DT*I
   17 WRITE(6,111) TT,RI(I),AF(I),D(I),SS(I)
      WRITE(6,120)
      WRITE(6,806)
  806 FORMAT(5X,'TIME FROM          MASS              MASS       DEPRESSION
     1DIFFERENCE',/,5X,'RAIN START   PRECIPITATED   INFILTRATED    STORA
     2GE      (MR-MI-DS)',/,5X,'(MINUTES)          (IN)            (IN)
     3     (IN)           (IN)    ',/)
      DO 807 I=1,N
      TT=DT*I
  807 WRITE(6,123) TT,RM(I),AFM(I),VS(I),PP(I)
  123 FORMAT(5X,F8.2,8X,F6.3,9X,F6.3,7X,F6.4,7X,F6.3)
      GO TO 118
C     IF INFILTRATION CAPACITY GREATER THAN RAINFALL INTENSITY
C     AT THE BEGINNING OF THE RAINFALL (WHERE TIME=O AND I=O) THEN
C     INFILTRATION CAPACITY CURVE STARTS AFTER A TIME (I*DT+X*L)FROM
C     THE BEGINNING OF THE RAINFALL
```

```
C      WE HAVE DIVIDED DT IN (NN+1) EQUAL INCREMENTS X(L FROM 1 TO NN)
C      THE SHIFTED INFILTRATION CAPACITY CURVE WILL INTERSECT THE
C      RAINFALL INTENSITY CURVE AFTER A TIME TJ=J*DT+Y AFTER THE
C      BEGINNING OF THE INFILTRATION CAPACITY CURVE.OR AFTER A TIME
C      (I*DT+X*L) FROM THE BEGINNING OF THE RAINFALL
C      THE I AFTER WHICH THE SECTION OF THE CURVES TAKES PLACE IS(ISEC)
   19 X=DT/AN
      NN=AN-1
      I=0
      DO 302 L=1,NN
      ALLL=(RO+(X*L*(RI(1)-RO)/DT)-FC)/(FO-FC)
      IF(ALLL.GT.0.0.AND.ALLL.LT.1.)GO TO 303
      GO TO 302
  303 TJ=-ALOG(ALLL)/CK
      B1=RO+(X*L*(RI(1)-RO)/(2.*DT))*X*L/60.
      B2=FC*TJ/60.+(FO-FC)*(1.-EXP(-CK*TJ))/(60.*CK)
      ISEC=I
      LSEC=L
      B12T=ABS(B1-B2)
      IF(B12T-0.01)301,301,302
  302 CONTINUE
      I=1
      DO 299 L=1,NN
      ALLL=((RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC))
      IF(ALLL.GT.0.AND.ALLL.LT.1.) GO TO 9999
      GO TO 299
 9999 TJ=-ALOG(ALLL)/CK
      B1=RM(I)+(RI(I)+(X*L*(RI(I+1)-RI(I))/(2.0*DT)))*(X*L/60.0)
      B2=FC*TJ/60.+(FO-FC)*(1.-EXP(-CK*TJ))/(60.*CK)
      ISEC=I
      LSEC=L
      B12T=ABS(B1-B2)
      IF(B12T-0.01) 301,301,299
  299 CONTINUE
      DO 300 I=2,N
      DO 300 L=1,NN
      RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      ALLL=(RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC)
      IF(ALLL.GT.0.AND.ALLL.LT.1.) GO TO 9998
      GO TO 300
 9998 TJ=-ALOG(ALLL)/CK
      B1=RM(I)+(RI(I)+(X*L*(RI(I+1)-RI(I))/(2.0*DT)))*(X*L/60.0)
      B2=FC*TJ/60.+(FO-FC)*(1.-EXP(-CK*TJ))/(60.*CK)
      ISEC=I
      LSEC=L
      B12T=ABS(B1-B2)
      IF(B12T-0.01) 301,301,300
  300 CONTINUE
C      BEFORE THE SECTION
  301 AFO=RO
      SO=0.0
      OD=0.0
      SSO=0.0
      DO 100 I=1,ISEC
      AF(I)=RI(I)
      S(I)=0.0
      D(I)=0.0
  100 SS(I)=0.0
C      AT THE SECTION
      IISEC=ISEC+1
      RISEC=(LSEC*X*(RI(IISEC)-RI(ISEC))/DT)+RI(ISEC)
      AFSEC=RISEC
      SSEC=0.0
      DSEC=0.0
      SSSEC=0.0
C     AFTER THE SECTION
      DO 101 I=IISEC,N
      AF(I)=FC+(FO-FC)*EXP(-CK*(TJ+DT*(I-ISEC)-X*LSEC))
  101 S(I)=RI(I)-AF(I)
C     BEFORE THE SECTION
      DO 102 I=2,ISEC
  102 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 103 I=1,ISEC
      AFM(I)=RM(I)
      VS(I)=0.0
      PP(I)=0.0
  103 P(I)=0.0
C     AT THE SECTION
      RMSEC=RM(ISEC)+((RI(ISEC)+RISEC)*X*LSEC)/120.
      AFMSE=RMSEC
      PSEC=0.0
      VSSEC=0.0
      PPSEC=0.0
C     AFTER THE SECTION
      DO 104 I=IISEC,N
  104 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 105 I=IISEC,N
  105 AFM(I)=(FC/60.0)*((I-ISEC)*DT-X*LSEC+TJ)+((FO-FC)/(60.0*CK))*(1.0-
     1EXP(-CK*((I-ISEC)*DT-X*LSEC+TJ)))
      DO 205 I=IISEC,N
  205 P(I)=RM(I)-AFM(I)
      NCON=0.0
      DO 810 I=IISEC,N
      IF(RI(I)-AF(I)) 901,901,795
  901 D(I)=0.0
      NCON=1.0
      VS(I)=SD
      SS(I)=S(I)
      PP(I)=P(I)-SD
      GO TO 810
  795 IF(NCON) 794,794,901
  794 D(I)=S(I)*EXP(-P(I)/SD)
      VS(I)=SD*(1.0-EXP(-P(I)/SD))
      SS(I)=S(I)-D(I)
      PP(I)=P(I)-VS(I)
  810 CONTINUE
      T=DT*ISEC+X*LSEC-TJ
      WRITE(6,106) T
  106 FORMAT(5X,'INFILTRATION CAPACITY CURVE STARTS AFTER THE',/,5X,'BEG
     1INNING OF THE  RAINFALL (IN MINUTES)  =',5X,F8.2,//)
      WTT=DT*ISEC+X*LSEC
      WRITE(6,108)WTT
  108 FORMAT(5X,'INFILTRATION CAPACITY CURVE INTERSECTS  THE',/,5X,'RAIN
     1FALL INTENSITY CURVE AFTER THE BEGINNING',/,5X,'OF THE RAINFALL
     2  (IN MINUTES)  =',12X,F8.2,//)
      WRITE (6,110)
  110 FORMAT(5X,'TIME FROM    RAINFALL        ACTUAL       DEPRESSION
     1OVERLAND ',/,5X,'RAIN START   INTENSITY   INFILTRATION   STOR.SUPP
     2LY   FLOWSUPPLY',/,5X,'(MINUTES)      (IN/HR)       (IN/HR)        (
     3IN/HR)       (IN/HR)  ',/)
```

```
      W=0.00
      WRITE(6,111)W,RO,AFO,DO,SSO
      DO 112 I=1,ISEC
      WW=DT*I
  112 WRITE(6,111)WW,RI(I),AF(I),D(I),SS(I)
      WRITE(6,111)WTT,RISEC,AFSEC,DSEC,SSSEC
      DO 115 I=IISEC,N
      WWW=DT*I
  115 WRITE(6,111) WWW,RI(I),AF(I),D(I),SS(I)
      WRITE(6,120)
      WRITE(6,121)
  121 FORMAT(5X,'TIME FROM         MASS             MASS       DEPRESSION
     1DIFFERENCE',/,5X,'RAIN START   PRECIPITATED   INFILTRATED    STORA
     2GE     (MR-MI-DS)',/,5X,'(MINUTES)          (IN)             (IN)
     3    (IN)          (IN)   ',/)
      DO 122 I=1,ISEC
      WW=DT*I
  122 WRITE(6,123) WW,RM(I),AFM(I),VS(I),PP(I)
      WRITE(6,123)WTT,RMSEC,AFMSE,VSSEC,PPSEC
      DO 125 I=IISEC,N
      WWW=DT*I
  125 WRITE(6,123) WWW,RM(I),AFM(I),VS(I),PP(I)
  118 WRITE (6,120)
C
C     ******************************************************
C     INFILTRATION AND DEPRESSION STORAGE IN IMPERVIOUS AREAS
C     ******************************************************
C
C     INFILTRATION CAPACITY IS ZERO
C     VOLUME OF WATER IN DEPRESSION STORAGE IS VSIM(I) INCHES
C     TOTAL DEPRESSION STORAGE CAPACITY IS SDIM (INCH)
C     DEPRESSION STORAGE SUPPLY IS DIM(I) INCH/HOUR
C     SSIM(I) IS THE RATE OF (RAINFALL-DEPR.STORAGE)
C     PPIM(I) IS THE ACCUMULATED MASS OF (RAIN-DEPR.STORAGE)
      WRITE(6,500)
  500 FORMAT(5X,'*****************************************************
     1********',/,5X,'SIMULATION OF INFILTR.AND DEPRESSION STORAGE IN IM
     2PERVIOUS AREA',/,5X,'*******************************************
     3******************',//)
      WRITE(6,25)
   25 FORMAT(5X,'INFILTRATION CAPACITY IS ZERO',//)
      WRITE(6,26)SDIM
   26 FORMAT(5X,'TOTAL DEPRESSION STORAGE CAPACITY OF THE',/,5X,'SUBCATC
     1HMENT IS    (IN INCHES)  =',15X,F8.4,//)
      DO 28 I=1,N
      CON1=EXP(-RM(I)/SDIM)
      DIM(I)=RI(I)*CON1
      VSIM(I)=SDIM*(1.-CON1)
      SSIM(I)=RI(I)-DIM(I)
      PPIM(I)=RM(I)-VSIM(I)
   28 CONTINUE
      WRITE(6,29)
   29 FORMAT(5X,'TIME FROM    RAINFALL    DEPRESSION    OVERL.FLOW',/,5X
     1,'RAIN START   INTENSITY   STOR.SUPPLY     SUPPLY',/,5X,'(MINUTES)
     2     (IN/HR)       (IN/HR)        (IN/HR)',/)
      TIMO=0.0
      DIMO=RO
      SSIMO=0.0
      WRITE(6,37)TIMO,RO,DIMO,SSIMO
   37 FORMAT(5X,F8.2,6X,F6.3,7X,F6.3,8X,F6.3)
   DO 30 I=1,N
   TIM=DT*I
30 WRITE(6,37) TIM,RI(I),DIM(I),SSIM(I)
   WRITE(6,120)
   WRITE(6,33)
33 FORMAT(5X,'TIME FROM        MASS       DEPRESSION   DIFFERENCE',/,
  15X,'RAIN START   PRECIPITATED    STORAGE      (RM-DS)',/,5X,'(MINU
  2TES)        (IN)          (IN)        (IN)',/)
   DO 34 I=1,N
   TTIM=DT*I
34 WRITE(6,35) TTIM,RM(I),VSIM(I),PPIM(I)
35 FORMAT(5X,F8.2,8X,F6.3,7X,F6.4,7X,F6.3)
   WRITE(6,120)
   CALL OVFLOW
   CALL GUTFL
   CALL PIROU
   CALL EXIT
   END
```

```
C
C     SUBROUTINE OVFLOW
C     *****************************
C     SIMULATION OF OVERLAND FLOW
C     *****************************
C
C     USE AS DATA THE OVERLAND FLOW SUPPLY FROM THE
C     INFILTRATION AND DEPRESSION STORAGE PROGRAM
C     ARW IS THE MANNING'S ROUGHNESS COEFFICIENT OF THE GROUND
C     ALW IS THE LENGTH OF THE OVERLAND FLOW IN FEET
C     SLW IS THE SLOPE OF THE SUBCATCHMENT IN THE DIRECTION OF FLOW
C     NW IS THE NUMBER OF THE SUBCATCHMENTS
C     INDW=1 INDICATES PERVIOUS SUBCATCHMENT,INDW=2 IMPERVIOUS
C     DE=DETENTION STORAGE AT EQUILIBRIUM (INCHES)
C     DC IS THE CURRENT DETENTION STORAGE(INCHES)
C     OVQ(I,IW) IS THE OVERLAND FLOW RATE OF THE SUBCATCHMENT OF
C     NUMBER IA,AT A TIME I*DT,IN (CFS/FT OF WIDTH)
C     OVQW(I) IS THE SAME OVERLAND FLOW IN(IN/HR/UNIT AREA)
      SUBROUTINE OVFLOW
      COMMON RI(120),AF(120),S(120),RM(120),AFM(120),P(120),D(120),
     1VS(120),SS(120),PP(120),SSIM(120),PPIM(120),VSIM(120),DIM(120),
     2DATE(8),DC(120),DE(120),OVQ(120,30),OVQW(120),NEW(30),QINL(120,40)
     3,QINLO(40),IW(600),V(600),N,DT,NN,NINL,QINP(600,40),QLEAV(600,40),
     4IUP(10),IDN(10),LNNEW(10),JPR,IPRINT
      READ(5,650) NW
  650 FORMAT(I5)
      DO 651 IA=1,NW
      READ(5,649)IA,INDW,ALW,ARW,SW
  649 FORMAT(2I2,3F7.4)
      WRITE(6,652)
  652 FORMAT(/,5X,'*****************************************************
     1**')
      IF(INDW-1) 653,654,653
  653 WRITE(6,655)IA
  655 FORMAT(5X,'OVERLAND FLOW SIMULATION OF IMPERVIOUS AREA NUMBER ',I2
     1)
      GO TO 657
  654 WRITE(6,656)IA
  656 FORMAT(5X,'OVERLAND FLOW SIMULATION OF  PERVIOUS AREA NUMBER ',I2)
  657 WRITE(6,658)
  658 FORMAT(5X,'*******************************************************
     1',//)
      WRITE(6,659)ALW
  659 FORMAT(5X,'LENGTH OF OVERLAND FLOW IN FEET         =',F7.2)
      WRITE(6,690) SW
  690 FORMAT(5X,'SLOPE AT THE DIRECTION OF FLOW (FT/FT) =',F7.4)
      WRITE(6,691) ARW
  691 FORMAT(5X,'MANNINGS  ROUGHNESS  COEFFICIENT        =',F7.4,//)
      AAA=510.35*SQRT(SW)/(ARW*ALW)
      CONS1=0.009792*(ARW*ALW/SQRT(SW))**0.6
      ABB=60.0/DT
      AB=1.6**1.6666
      BA=AAA**0.6
      IF(IPRINT.EQ.0)GO TO 503
      WRITE(6,692)
  692 FORMAT(5X,'TIME FROM       OVERLAND FLOW AT       OVERLAND FLOW AT',
     1/,5X,'RAIN START      THE DOWNSTREAM          THE DOWNSTREAM',/,5X,'
     2 IN           END OF WATERSHED      END OF WATERSHED',/,5X,' MINUT
     3ES       (IN/HR/UNIT AREA)      (CFS/FT OF WIDTH)',/)
  503 IV=IISEC
```

```
      OVV=WTT
      IF(INOW-1) 680,681,680
  680 WTT=0.0
      IISEC=1
      DO 682 I=1,N
  682 SS(I)=SSIM(I)
      GO TO 683
  681 IF(RO-FO) 683,684,684
  684 WTT=0.0
      IISEC=1
  683 DESEC=0.0
      OVOO=0.0
      OVOWO=0.0
      IF(IPRINT.EQ.0)GO TO 504
      WRITE(6,850)WTT,OVOWO,OVOO
  850 FORMAT(7X,F6.2,11X,F7.4,13X,F9.6)
  504 IF(IISEC-1) 678,677,678
  678 DO 849 I=1,IISEC
  849 OVO(I,IA)=0.0
  677 DO 660 I=IISEC,N
      IF(SS(I)) 6659,6659,6600
 6659 DE(I)=0.0
      GO TO 660
 6600 DE(I)=CONS1*SS(I)**0.6
  660 CONTINUE
      I=IISEC
      IF(SS(I)-0.02)830,830,840
  830 DC(I)=0.0
      OVOW(I)=0.0
      OVO(I,IA)=0.0
      GO TO 829
  840 STA=0.5*SS(I)
      AA=0.6/(DE(I)**3)
      ABA=AA*BA
      DTO=(DT*I)-WTT
      BBA=60.0/DTO
      G1=0.5
      DO 851 ID=1,40
      CONS2=ABA*G1**4
      CONS3=BA*G1+CONS2
      CONS4=CONS3**1.6666
      F1=CONS4+BBA*G1-STA
      F2=1.6666*(BA+4.*CONS2/G1)*CONS4/CONS3+BBA
      G=G1-F1/F2
      BMA=ABS(G1-G)
      IF(BMA.LT.0.0001)GO TO 852
      G1=G
  851 CONTINUE
      DC(I)=G
      OVOW(I)=2.*AAA*(DC(I)*(1.+AA*DC(I)**3))**1.6666
      OVO(I,IA)=ALW*OVOW(I)/43200.0
  829 TIME=DT*I
      WRITE(6,853) TIME,OVOW(I),OVO(I,IA),DE(I),DC(I),SS(I)
  853 FORMAT(7X,F6.2,11X,F7.4,13X,F9.6,10X,F8.5,10X,F8.5,10X,F6.3)
      NWN=N+8
      NSTAR=N+1
      DO 661 I=NSTAR,NWN
      SS(I)=SS(N)
  661 DE(I)=DE(N)
      KKW=NWN-1
      SUM=0.0
      DO 663 I=IISEC,KKW
      STA=0.5*(SS(I)+SS(I+1)-OVQW(I))+ABB*DC(I)
      K=I+1
      IF(STA) 1652,1652,9888
 9888 IF(DE(K)-0.001) 666,662,662
  662 AA=0.6/(DE(K)**3)
      ABA=AA*BA
      G1=0.5
      DO 646 ID=1,40
      CONS2=ABA*G1**4
      CONS3=BA*G1+CONS2
      CONS4=CONS3**1.6666
      F1=CONS4+ABB*G1-STA
      F2=1.6666*(BA+4.*CONS2/G1)*CONS4/CONS3+ABB
      G=G1-F1/F2
      BMA=ABS(G1-G)
      IF(BMA.LT.0.00001)GO TO 665
      G1=G
  646 CONTINUE
      WRITE(6,502)G,G1
  502 FORMAT(/,2X,'**** NO CONVERGENCE **** G=',F10.6,' G1=',F10.6,/)
  665 DC(K)=G
      IF(DE(K).GE.DC(K))GO TO 667
      G1=0.5
      DO 644 ID=1,40
      CONS2=AAA*AB*G1**1.6666
      CONS3=ABB*G1
      F1=CONS2+CONS3-STA
      F2=(1.6666*CONS2+CONS3)/G1
      G=G1-F1/F2
      BMA=ABS(G1-G)
      IF(BMA.LT.0.00001)GO TO 669
      G1=G
  644 CONTINUE
      WRITE(6,502)G,G1
  669 DC(K)=G
      GO TO 676
 1652 DC(K)=0.0
      OVQW(K)=0.0
      GO TO 670
  676 OVQW(K)=2.*AAA*AB*DC(K)**1.6666
      GO TO 670
  667 OVQW(K)=2.*AAA*(DC(K)*(1.+AA*(DC(K)**3)))**1.6666
  670 OVO(K,IA)=ALW*OVQW(K)/43200.0
      TIME=K*DT
      IF(IPRINT.EQ.0)GO TO 505
      WRITE(6,853) TIME,OVQW(K),OVO(K,IA),DE(K),DC(K),SS(K)
  505 SUM=SUM+(OVQW(I)+OVQW(K))*DT/120.
      IF(K-N) 663,672,672
  672 IF(OVQW(K)-0.0009) 673,663,663
  663 CONTINUE
  673 NEW(IA)=K
      NNNN=NEW(IA)+1
      DO 6556 I=NNNN,NWN
      OVO(I,IA)=0.0
 6556 CONTINUE
      WRITE(6,674) SUM
  674 FORMAT(//,5X,'TOTAL VOLUME(INCHES)WHICH RAN-OFF IS =',F7.3)
      IISEC=IV
      WTT=OVV
      WRITE(6,120)
  120 FORMAT('1')
  651 CONTINUE
      RETURN
      END
```

```
C
C      SUBROUTINE GUTFL
C      **********************
C      GUTTER  FLOW  PROGRAM
C      **********************
C
C
C      NINL IS THE NUMBER OF INLETS
C      N IS THE NUMBER OF TIME INCREMENTS
C      NW IS THE NUMBER OF SUBCATCHMENTS
C      THE PROGRAM WILL TREAT INLETS ACCEPTING A NUMBER OF
C      NBRAN  SYSTEMS OF GUTTERS,AND DIRECT CONNECTIONS
C      N1 IS THE NUMBER OF THE FIRST UPSTREAM GUTTER OF
C      EACH GUTTER SYSTEM
C      N2 IS THE NUMBER OF THE LAST DOWNSTREAM GUTTER OF
C      EACH GUTTER SYSTEM
C      ITYPL IS THE NUMBER OF SUBCATCHMENT FEEDING  A  GUTTER
C      FROM THE LEFT SIDE
C      ITYPR IS THE NUMBER OF THE SUBCATCHMENT FEEDING  A  GUTTER
C      FROM THE RIGHT SIDE
C    ' GLEN IS THE LENGTH OF THE GUTTER
C      IROOF IS THE NUMBER OF HOUSE ROOFS DIRECTLY CONNECTED WITH AN INLET
C      IWRF IS THE NUMBER OF THE TYPICAL HOUSE ROOF SUBCATCHMENT
C      ROOFL IS THE WIDTH OF A ROOF IN FEET
C      NZZ1=2 IF DIRECT CONNECTIONS EXIST,OTHERWISE NZZ1=1
C      NMAXI=MAXIMUM NUMBER OF TIME INTERVALS(SEE DIMENSION STAT.)
       SUBROUTINE GUTFL
       COMMON RI(120),AF(120),S(120),RM(120),AFM(120),P(120),D(120),
      1VS(120),SS(120),PP(120),SSIM(120),PPIM(120),VSIM(120),DIM(120),
      2DATE(8),DC(120),DE(120),OVQ(120,30),OVQW(120),NEW(30),QINL(120,40)
      3,QINLO(40),IW(600),V(600),N,DT,NN,NINL,QINP(600,40),QLEAV(600,40),
      4IUP(10),IDN(10),LNNEW(10),JPR,IPRINT
       READ(5,3030)NZZ1,NMAXI
 3030  FORMAT(2I5)
       READ(5,3000) NINL
 3000  FORMAT(I5)
       DO 3004 IIN=1,NINL
       DO 3004 I=1,NMAXI
       QLEAV(I,IIN)=0.0
       QINP(I,IIN)=0.0
 3004  QINL(I,IIN)=0.0
       DO 4001 IIN=1,NINL
       READ(5,3009) NBRAN
 3009  FORMAT(I5)
       IF(NBRAN.EQ.0)GO TO 4019
       DO 4010 I1=1,NBRAN
       READ(5,3010) N1,N2
 3010  FORMAT(2I5)
       DO 4002 J=N1,N2
       READ(5,3011) ITYPL,ITYPR,GLEN
 3011  FORMAT(2I5,F10.2)
       IF(ITYPR.EQ.0.AND.ITYPL.EQ.0)GO TO 4002
       IF(ITYPR.NE.0)GO TO 4004
       DO 4003 I=1,N
       QGUT=OVQ(I,ITYPL)*GLEN
 4003  QINL(I,IIN)=QINL(I,IIN)+QGUT
       GO TO 4002
 4004  IF(ITYPL.NE.0)GO TO 4006
       DO 4007 I=1,N
       QGUT=OVQ(I,ITYPR)*GLEN
 4007  QINL(I,IIN)=QINL(I,IIN)+QGUT
```

```
      GO TO 4002
 4006 DO 4012 I=1,N
      OGUT=(OVQ(I,ITYPL)+OVQ(I,ITYPR))*GLEN
 4012 QINL(I,IIN)=QINL(I,IIN)+OGUT
 4002 CONTINUE
 4010 CONTINUE
 4019 GTIMO=0.0
      QINLO(IIN)=0.0
      IF(NZZ1.EQ.1)GO TO 4001
      READ(5,3013) IROOF,IWRF,ROOFL
 3013 FORMAT(2I5,F5.2)
      DO 4005 I=1,N
C     HERE OGUT SHOUS FLOW ENTERING THE INLET FROM DIRECT CONNECTIONS
      OGUT=IROOF*2.0*ROOFL*OVQ(I,IWRF)
      QINL(I,IIN)=QINL(I,IIN)+OGUT
 4005 CONTINUE
 4001 CONTINUE
      IF(IPRINT.EQ.0)GO TO 5950
    . DO 5890 NINI=1,10
      WRITE(6,5500)
 5500 FORMAT('1')
      N15=10*NINI-9
      N16=10*NINI
      WRITE(6,5555)(I,I=N15,N16)
 5555 FORMAT(2X,'TIME FROM',10X,'DISCHARGE IN CUB.FEET/SEC ENTERING THE
     1INLETS OF NUMBER:',/2X,'RAIN START',3X,I2,9I10,/4X,'(MIN)',//)
      IF(N16-NINL) 5800,5800,5810
 5810 N16=NINL
 5800 WRITE(6,3014) GTIMO,(QINLO(IIN),IIN=N15,N16)
 3014 FORMAT(3X,F6.2,10F10.3)
      DO 5850 I=1,N
      GTIM=I*DT
      WRITE(6,3014) GTIM,(QINL(I,J),J=N15,N16)
 5850 CONTINUE
      IF(N16-NINL) 5890,5950,5950
 5890 CONTINUE
 5950 DDD=1.0
C
C     ***********************************************************
C     PROGRAM TO TRANSFORM HYDROGRAPHS INTO ONE MIN.INCREMENTS
C     ***********************************************************
C
C     THIS IN ORDER TO FACILITATE THE ADDITION OF THE HYDROGRAPHS
C     DURING THE ROUTING THROUGH THE LATERAL AND MAIN SEWERS
      KK=DT/DDD
      NN=KK*N
      DO 6901 IIN=1,NINL
      DO 6901  I=1,N
      JJ=N+2-I
      JJJ=N+1-I
      QINL(JJ,IIN)=QINL(JJJ,IIN)
 6901 CONTINUE
      DO 6900 IIN=1,NINL
 6900 QINL(1,IIN)=0.0
      DO 6912 IIN=1,NINL
      II=0
      DO 6911 I=1,N
      DO 6911 K=1,KK
      II=II+1
      QINP(II,IIN)=QINL(I,IIN)+((2*K-1)*(DDD/(2.0*DT))*(QINL(I+1,IIN)-QI
     1NL(I,IIN)))
 6911 CONTINUE
 6912 CONTINUE
      IF(IPRINT.EQ.0)GO TO 6923
      DO 6922 NINI=1,10
      WRITE(6,5500)
      N15=10*NINI-9
      N16=10*NINI
      WRITE(6,5555)(I,I=N15,N16)
      IF(N16-NINL) 6918,6918,6919
 6919 N16=NINL
 6918 DO 6920 I=1,NN
      IF(I.EQ.1)GO TO 6924
      IF(I/JPR*JPR.NE.I)GO TO 6920
 6924 TRTIM=DDD*(I-1)
      WRITE(6,3014) TRTIM,(OINP(I,J),J=N15,N16)
 6920 CONTINUE
      IF(N16-NINL) 6922,6923,6923
 6922 CONTINUE
 6923 RETURN
      END
```

```
C
C     SUBROUTINE PIROU
C     *****************************************
C     ROUTING THROUGH LATERAL AND MAIN SEWERS
C     *****************************************
C
C     USE OF MANNING'S EQUATION TO FIND THE AVERAGE TIME NEEDED
C     FOR A DISCHARGE HYDROGRAPH TO TRAVEL DOWNSTREAM UNTIL A
C     POINT(INLET,MANHOLE)WHERE A SECOND DISCHARGE HYDROGRAPH
C     ENTERS THE SEWER SYSTEM.ADD THE SHIFFTED AND THE NEW
C     HYDROGRAPH AND CONTINUE UNTIL THE POINT FOR WHICH WE SEEK
C     THE OUTFLOW HYDROGRAPH
C     'AREA' IS THE AREA OF FLOW IN SQUARE FEET
C     'SLOP' IS THE SLOPE OF THE INVERT IN FT/FT
C     'ALEN' IS THE LENGTH BETWEEN TWO CRITICAL POINTS(INLETS,
C     MANHOLES.CHANGE IN SLOPE OR DIAMETER OR SHAPE)
C     (WHEN THERE IS A CHANGE IN SLOPE OR DIAMETER OR SHAPE WE
C     CONSIDER THERE AN HYPOTHETICAL MANHOLE WITH ZERO INFLOW)
C     'PIPN' IS THE MANNING'S ROUGHNESS COEFFICIENT OF THE PIPE
C     'Y' IS THE DEPTH OF FLOW IN THE CONDUIT IN FEET
C     'HR' IS THE HYDRAULIC RADIUS IN FEET
C     'DIA' IS THE DIAMETER OF THE SEWER PIPE IN FEET OR ITS WIDTH
C     IF IT IS RECTANGULAR
C     'H'  IS THE HEIGHT OF RECTANGULAR SEWER IN FEET
C     I4=1 INDICATES CIRCULAR PIPE,I4=2 RECTANGULAR CONDUIT
C     NINL=NUMBER OF INLETS,MANHOLES AND CHANGES IN SLOPE AND DIAMETER
C     NPIPE=NUMBER OF PIPES
C     NUMIN=NUMBER OF DOWNSTREAM INLET
C     NUMOUT=NUMBER OF UPSTREAM INLET
C     NCONEC=1 IF BRANCHING AT DOWNSTREAM END OF PIPE
C           =2 IF PIPELINE CONTINUES WITHOUT BRANCHING
C           =3 IF WE MEET BRANCH INLET FOR SECOND TIME
C     INDWT=NUMBER OF BRANCHES MET BEFORE TO RETURN AT THE
C           INLET UNDER CONSIDERATION
      SUBROUTINE PIROU
      COMMON RI(120),AF(120),S(120),RM(120),AFM(120),P(120),D(120),
     1VS(120),SS(120),PP(120),SSIM(120),PPIM(120),VSIM(120),DIM(120),
     2DATE(8),DC(120),DE(120),QVQ(120,30),QVQW(120),NEW(30),QINL(120,40)
     3,QINLO(40),IW(600),V(600),N,DT,NN,NINL,QINP(600,40),QLEAV(600,40),
     4IUP(10),IDN(10),LNNEW(10),JPR,IPRINT
      READ(5,909) NPIPE
  909 FORMAT(I5)
      NIN=NINL-1
      DO 1111 IK=1,10
 1111 IUP(IK)=0
      DO 1050 IIN=1,NIN
      J=IIN
      WRITE(6,917)
  917 FORMAT('1')
      READ(5,907) I4,SLOP,PIPN,DIA,' ,ALEN,NCONEC,NUMOUT,NUMIN,INDWT
  907 FORMAT(I5,F10.6,F10.4,2F5.3,F10.2,4I5)
      KK=DT
      NN=KK*N
      SQSL=SQRT(SLOP)
      IF(I4-1) 950,905,950
  905 CON2=DIA**2.6666*SQSL/PIPN
      QMAX=0.46417*CON2
      CCC=13.566/CON2
      QHALF=0.5*QMAX
      GO TO 199
```

```
950 CON3=PIPN/(1.486*SQSL*DIA**1.6666)
    QMAX=(H/(DIA+2.*H))**0.6666*H/CON3
199 SPROD=0.0
    SDIS=0.0
    DO 200 I=1,NN
    TI=FLOAT(I)
    IW(I)=0
    IF(QINP(I,IIN))200,200,201
201 IF(QMAX-QINP(I,IIN))202,203,203
202 IW(I)=1
203 SPROD=SPROD+TI*QINP(I,IIN)
    SDIS=SDIS+QINP(I,IIN)
200 CONTINUE
    IF(SDIS.LE.0.001)GO TO 52
    CDIS=SPROD/SDIS
    ICEN=IFIX(CDIS+0.5)
    QCEN=QINP(ICEN,IIN)
    IF(QCEN.LT.QMAX)GO TO 50
    WRITE(6,51)QCEN,QMAX
 51 FORMAT(/,'  QCEN=',F8.2,'  QMAX=',F8.2,'  X EQUALED TO=6.28')
 52 QCEN=QMAX
    IF(I4.EQ.1)X=6.28318
    IF(I4.EQ.2)Y=H
    GO TO 205
 50 IF(I4.EQ.2)GO TO 210
    IF(QCEN.GT.QHALF)X1=4.5
    IF(QCEN.LE.QHALF)X1=1.5
    CO1=CCC*QCEN
    DO 204 K=1,60
    IF(X1.GT.6.28318)X1=6.28318
    B=(X1-SIN(X1))**1.6666
    BB=X1**0.6666
    X=X1-(B/BB-CO1)/((5.*X1*B**.4*(1.-COS(X1))-2.*B)/(3.*BB*X1))
    IF(X.LT.0.0)X=-X
    IF(ABS(X-X1).LT.0.005)GO TO 205
    X1=X
204 CONTINUE
    WRITE(6,502)X,X1
502 FORMAT(/,'  **** NO CONVERGENCE ****  X=',F10.6,'  X1=',F10.6,/)
    X=6.28318
    GO TO 205
210 Y1=0.5*H
    B=CON3*QCEN
    DO 959 K=1,40
    BB=(Y1/DIA+2.*Y1)**0.6666*Y1
    Y=Y1-(BB-B)/(1.6666*BB/Y1-1.3333*BB/(DIA+2.*Y1))
    IF(ABS(Y-Y1).LT.0.001)GO TO 205
    Y1=Y
959 CONTINUE
    WRITE(6,502)Y,Y1
    Y=H
205 IF(I4.EQ.2)GO TO 207
    VEL=8.*QCEN/(DIA**2*(X-SIN(X)))
    GO TO 208
207 VEL=QCEN/(DIA*Y)
208 DO 206 I=1,NN
206 V(I)=VEL
    AVERV=VEL
    AVT=ALEN/(AVERV*60.0)
    IAVET=IFIX(AVT+0.5)
```

```
      AVERT=FLOAT(IAVET)
      IIAVT=IAVET+1
      NNEW=NN+IAVET
      NEEW=NN+1
      DO 995 I=IIAVT,NNEW
      KL=I-IAVET
  995 QLEAV(I,IIN)=QINP(KL,IIN)
      DO 996 I=NEEW,NNEW
  996 IW(I)=0
      WRITE(6,997) J,NUMOUT,NUMIN
  997 FORMAT(28X,'*****************',/,28X,'PIPE  NUMBER  ',I2,/,28X,'***
     1**************',//,5X,'UPSTREAM INLET NUMBER=',I2,'  DOWNSTREAM INL
     2ET NUMBER=',I2)
      IF(I4-1) 968,969,968
  969 WRITE(6,967) DIA
  967 FORMAT(5X,'CIRCULAR CONDUIT  OF  DIAMETER   (FEET)
     1  ',F5.2)
      GO TO 962
  968 WRITE(6,966) DIA,H
  966 FORMAT(5X,'RECTANGULAR  CONDUIT  WITH  DIMENSIONS  IN FEET    ',F5
     1.2,'*',F5.2)
  962 WRITE(6,965) SLOP
  965 FORMAT(5X,'THE SLOPE OF THE PIPE INVERT IN FT/FT  IS
     1',F7.5)
      WRITE(6,964) ALEN
  964 FORMAT(5X,'THE LENGTH   OF  THE  PIPE  IN  FEET  IS
     1',F7.2)
      WRITE(6,963) PIPN
  963 FORMAT(5X,'THE  MANNINGS  ROUGHNESS  COEFFICIENT  IS
     1',F7.5)
      WRITE(6,998)QMAX
  998 FORMAT(5X,'MAXIMUM CAPACITY OF THE CONDUIT IS (CU.FEET/SEC)
     1 ',F7.2)
      WRITE(6,999) AVERV
  999 FORMAT(5X,'AVERAGE VELOCITY OF ENTERING DISCHARGES (FT/SEC)
     1  ',F5.2)
      WRITE(6,1001) AVERT
 1001 FORMAT(5X,'AVERAGE TIME OF TRAVEL OF THE ENTERING HYDROGRAPH(MIN)
     1  ',F7.2,//)
      WRITE(6,1002)
 1002 FORMAT(5X,'TIME FROM      DISCHARGE               DISCHARGE',/,5X,
     1'RAIN START     ENTERING                LEAVING THE',/,5X,'(MINUTES)
     2   PIPE (CFS)             PIPE (CFS)',/)
      USM=0.0
      IF(IIN-NIN) 1069,1078,1078
 1078 DO 1074 I=1,NNEW
      QLEAV(I,NIN)=QLEAV(I,NIN)+QINP(I,NINL)
 1074 USM=USM+QLEAV(I,NIN)*60.
 1069 DO 1003 I=1,NNEW
      IF(I.EQ.1)GO TO 1068
      IF(I/JPR*JPR.NE.I)GO TO 1003
 1068 TIMP=I-1
      IF(IW(I)-1) 1072,1080,1072
 1072 WRITE(6,1004) TIMP,QINP(I,IIN),QLEAV(I,IIN)
 1004 FORMAT(6X,F7.2,7X,F8.3,13X,F8.3)
      GO TO 1003
 1080 WRITE(6,1005) TIMP,QINP(I,IIN)
 1005 FORMAT(6X,F7.2,7X,F8.3,7X,'CONDUIT OVERFLOWS')
 1003 CONTINUE
      IF(IIN.NE.NIN)GO TO 1057
      USM=USM/1000.
      WRITE(6,1058)USM
 1058 FORMAT(/,5X,'TOTAL VOLUME OF WATER DISCHARGED FROM THE OUTLET IN T
     1HOUSANDS CUBIC FEET IS=',F15.3)
      GO TO 1050
 1057 IF(NCONEC.NE.1)GO TO 1052
      I1=INDWT
      IUP(I1)=IIN
      IDN(I1)=NUMIN
      LNNEW(I1)=NNEW
      GO TO 1050
 1052 IF(NCONEC.NE.2)GO TO 1053
      DO 1021 I=1,NNEW
 1021 QINP(I,IIN+1)=QINP(I,IIN+1)+QLEAV(I,IIN)
      GO TO 1050
 1053 DO 1056 IK=1,10
      IF(IUP(IK).GT.0)GO TO 1055
 1056 CONTINUE
 1055 IF(LNNEW(IK).GT.NNEW)NNEW=LNNEW(IK)
      DO 1054 I=1,NNEW
 1054 QINP(I,IDN(IK))=QINP(I,IDN(IK))+QLEAV(I,IUP(IK))+QLEAV(I,IIN)
      IUP(IK)=0
 1050 CONTINUE
      RETURN
      END
```

APPENDIX B

DATA CARD PREPARATION FOR THE UCUR MODEL

A. Rainfall Data Cards

I. card 1 (8A4, F5.2, 15)

			Variable Names	Example Values
col.	1-20	Date of storm (day, month, year)	DATE	24 SEPT 1970
col.	21-32	Hour of start of storm (hours, min., AM or PM)	DATE	5:35 AM
col.	33-37	Time interval between intensity values (minutes)	DT	5.00
col.	38-42	Number of time steps to be calculated	N	36
col.	43-48	If reduced printout is desired set I PRINT = 0	I PRINT	0

II. card 2 (F5.2)

col.	1-5	First value of rainfall intensity	RO	0.20

III. Rainfall intensity cards, each (12F5.2)

Rainfall intensity in in/hr (N values)	RI	

B. Infiltration Data Cards

I. card 1 (3F10.5, 2F8.4)

col.	1-10	Const. infiltr. rate (in/hr)	FC	0.53
col.	11-20	Initial infiltr. rate (in/hr)	FO	3.00
col.	21-30	Decay rate of infil. (1/min.)	CK	0.0697
col.	31-38	Retention storage of pervious areas (in)	SD	0.250
col.	39-46	Retention storage of impervious areas (in)	SDIM	0.0625

		Variable Names	Example Values
II. card 2 (F5.2,I5)			
col. 1-5	A number to divide DT in smaller intervals (see statement 19. If DT = 5.0 min, a logical value of AN is 20.0 or 30.0. Then the new intervals will be of 0.17 min. each)	AN	30.0
col. 6-10	Print out obtained every JPR times from GUTFL and PIROU	JPR	1
C. Overland Flow Data Cards			
I. card 1 (I5)			
col. 1-5	Number of subcatchments to be simulated	NW	6
II. One card for each subcatchment (2I2, 3F7.4) (pervious subcatchments first)			
col. 1-2	Number of the subcatchment	IA	1
col. 3-4	Indication number (1 for pervious, 2 for impervious)	INDW	1
col. 5-11	Length of area at the direction of flow (feet)	ALW	16.0
col. 12-18	Manning's roughness coef.	ARW	0.35
col. 19-25	Slope of the area at the direction of flow (ft/ft)	SW	0.01
D. Gutters Data Cards			
I. card 1 (2I5)			
col. 1-5	If direct connections exist NZZ1 = 2, otherwise NZZ1 = 1	NZZ1	2
col. 6-10	Maximum number of time intervals (Larger than N)	NMAXI	50
II. card 2 (I5)			
col. 1-5	Number of entry points into the sewer system	NINL	9

(Data cards of type III, IV, V and VI to be repeated for each entry point)

		Variable Names	Example Values
III. card 3 (I5)			
col. 1-5	Number of gutter branches leading into the entry point	NBRAN	2
(Data cards of type IV, V to be repeated for each gutter branch if NBRAN is not zero. If NBRAN = 0 there are no gutters conducting flow into the particular inlet. In this case disregard data cards of type IV and V)			
IV. card 4 (2I5)			
col. 1-5	Number of the first upstream gutter segment of the gutter branch	N1	1
col. 6-10	Number of the last downstream gutter segment of the gutter branch	N2	2
(Data cards of type V to be repeated for each gutter segment)			
V. card 5 (2I5, F10.2)			
col. 1-5	Number of the subcatchment providing flow to the gutter segment from the left	ITYPL	3
col. 6-10	Number of the subcatchment providing flow to the gutter segment from the right	ITYPR	5
(If there is no inflow in a gutter from the left or right, set ITYPL or ITYPR equal to zero).			
col. 11-20	Length of the gutter segment in feet	GLEN	297.0
VI. card 6 (2I5, F5.2)			
col. 1-5	Number of roofs directly connected with entry point	IROOF	0
col. 6-10	Number of subcatchment corresponding to the roof type	IWRF	6
col. 11-15	Length of the roofs (ft)	ROOFL	30.0
(Omit data cards of type VI if NZZ1 = 1)			

E. Pipes Data Cards

			Variable Names	Example Values
I.	card 1 (I5)			
	col. 1-5	Number of pipes	NPIPE	8
II.	one card for each pipe (I5, F10.6, F10.4, 2F5.2, F10.3,4I5)			
	col. 1-5	Indication number, 1 for circular pipe, 2 for rectangular	I4	1
	col. 6-15	Slope of the pipe invert	SLOP	0.0045
	col. 16-25	Manning's roughness coefficient	PIPN	0.013
	col. 26-30	Diameter of circular pipe or width of rectangular pipe (ft)	DIA	3.0
	col. 31-35	Height of rectangular pipe (0.0 for circular) (ft)	H	0.0
	col. 36-45	Length of pipe (ft)	ALEN	120.0
	col. 46-50	If branching at downstream end of pipe, NCONEC = 1. If pipeline continues without branching, NCONEC = 2. If a branch inlet is met for second time NCONEC = 3.	NCONEC	2
	col. 51-55	Number of upstream inlet	NUMOUT	5
	col. 56-60	Number of downstream inlet	NUMIN	6
	col. 61-65	Number of branches met before returning to the inlet under consideration	INDWT	0

APPENDIX C

EXPLANATION OF CODING SYSTEM
EPA STORM WATER MANAGEMENT MODEL

The input variables below are coded with numbers (0, 1, 2, etc.) denoting changes in various sets of data or parameters deemed important at the time of testing. The key to these numbers is as follows:

Storm date:	Indicates a particular set of hyetographs and related storm data. This is the most important variable.
GP-O:	No "gutter/pipe" cards were used.
GP-1:	"Gutter/pipe" cards were used for 3 large subcatchments only (Bloody Run tests).
GP-2:	"Gutter/pipe" cards were used for all but the 3 smallest subcatchments of Bloody Run.
GP-3:	"Gutter/pipe" cards were supplied incorrectly to program due to misinterpretation of User's Manual. This had the same effect on the output as did GP-0. (Bank tests)
GP-4:	"Gutter/pipe" cards were used for 92 of the subcatchments of the Bank Watershed. See card group 7, Runoff Block p. 60 of the User's Manual for description of "gutter/pipe" data.
DWF data-1:	Dry weather flow data was omitted for 47 of the 71 subareas of Bloody Run
DWF data-2:	Dry weather flow data was included for all 71 subareas of Bloody Run
DWF data-3:	Dry weather flow data was included for 3 subareas of the Bank Watershed. This group is always associated with GP-O and GP-3. (i.e. no "gutter/pipes")
DWF data-4:	Dry weather flow data was included for the 95 subareas of the Bank Watershed as "gutter/pipe" information was now included. This group is always associated with GP-4. Refer to the description of card group 44 in the Transport Block of the model (see User's Manual, p. 180) for further information.

PCTZER-1: Percent of impervious area having zero detention = 25.0 (the program's default value).

PCTZER-2: Percent of impervious area having zero detention = 42.0 (the actual Bloody Run value).

PCTZER-3: Percent of impervious area having zero detention = 40.0 (the actual Bank Watershed value).

PCTZER-4: Percent of impervious area having zero detention = 1.0 (an experimental value). Refer to card group 2, Runoff Block, p. 59 in User's Manual for further information.

INFILTRATION data-1: The maximum and minimum infiltration rates were assumed to be 0.01. The decay rate was set at 0.00113.

INFILTRATION data-2: The program's default values of 3.00 (max. rate), 0.52 (min. rate) and 0.00115 (decay rate) were used.

INFILTRATION data-3: The maximum infiltration rate was set at 4.75, the minimum rate at 0.44, and the decay rate at 0.00281 in accordance with data collected from the Bloody Run Watershed. This may also apply to Bank testing. (See card group 5, Runoff Block, p. 60 of User's Manual for further description of these parameters.)

DINFIL-1: Base dry weather infiltration = 2550 gpm.

DINFIL-2: Base dry weather infiltration = 100 gpm. (See card group 30, Transport Block, p. 177 in the User's Manual for details on this factor).

TABLE A-1

SUMMARY OF COMPUTER TESTS USING THE EPA SWMM

a) BLOODY RUN TESTING

Comparison Number	Description of Input and Comparisons
	Storm of 10/20/70; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-1; DINFIL-1.
	Hydrographs compared at
1	OUTLET
	Suspended solids pollutographs compared at
2	OUTLET
3	BANK 54"
	BOD pollutographs compared at
4	OUTLET
5	BANK 54"
	Storm of 10/20/70; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
6	OUTLET
	Storm of 11/9/70; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
7	OUTLET
8	12' x 9'
9	11' x 9'
10	BANK 78"
	Suspended solids pollutographs compared at
11	BANK 78"
12	BANK 54"
	BOD pollutographs compared at
13	12' x 9'
14	11' x 9'
15	BANK 78"
16	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Storm of 11/9/70; GP-2; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
17	OUTLET
18	12' x 9'
19	11' x 9'
20	BANK 78"
	Suspended solids pollutographs compared at
21	BANK 78"
22	BANK 54"
	BOD pollutographs compared at
23	12' x 9'
24	11' x 9'
25	BANK 78"
26	BANK 54"
	Storm of 11/14/70; GP-0; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
27	OUTLET
28	12' x 9'
29	11' x 9'
30	BANK 78"
	Suspended solids pollutographs compared at
31	BANK 78"
32	BANK 54"
	BOD pollutographs compared at
33	12' x 9'
34	11' x 9'
35	BANK 78"
36	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Storm of 11/14/70; GP-1; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
37	OUTLET
38	12' x 9'
39	11' x 9'
40	BANK 78"
	Suspended solids pollutographs compared at
41	BANK 78"
42	BANK 54"
	BOD pollutographs compared at
43	12' x 9'
44	11' x 9'
45	BANK 78"
46	BANK 54"
	Storm of 11/14/70; GP-2; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
47	OUTLET
48	12' x 9'
49	11' x 9'
50	BANK 78"
	Suspended solids pollutographs compared at
51	BANK 78"
52	BANK 54"
	BOD pollutographs compared at
53	12' x 9'
54	11' x 9'
55	BANK 78"
56	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Storm of 5/7/71; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
57	OUTLET
58	12' x 9'
59	11' x 9'
60	BANK 78"
61	BANK 54"
	Suspended solids pollutographs compared at
62	OUTLET
63	12' x 9'
	BOD pollutographs compared at
64	OUTLET
65	12' x 9'
	Storm of 5/7/71; GP-2; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
66	OUTLET
67	12' x 9'
68	11' x 9'
69	BANK 78"
70	BANK 54"
	Suspended solids pollutographs compared at
71	OUTLET
72	12' x 9'
	BOD pollutographs compared at
73	OUTLET
74	12' x 9'

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Storm of 5/13/71; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
75	OUTLET
76	12' x 9'
77	11' x 9'
78	BANK 78"
79	BANK 54"
	Suspended solids pollutographs compared at
80	OUTLET
	BOD pollutographs compared at
81	OUTLET
	Storm of 5/13/71; GP-2; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
82	OUTLET
83	12' x 9'
84	11' x 9'
85	BANK 78"
86	BANK 54"
	Suspended solids pollutographs compared at
87	OUTLET
	BOD pollutographs compared at
88	OUTLET
	Storm of 5/13/71; GP-0; DWF data-2; PCTZER-2; INFILTRATION data-3; DINFIL-1.
	Hydrographs compared at
89	OUTLET
90	12' x 9'
91	11' x 9'
92	BANK 78"
93	BANK 54"

TABLE A-1 (Continued)

Comparison Number	Description of Input and Comparisons
	Suspended solids pollutographs compared at
94	OUTLET
	BOD pollutographs compared at
95	OUTLET
	Storm of 8/25-6/71; GP-0; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
96	OUTLET
97	11' x 9'
98	BANK 78"
99	BANK 54"
	Suspended solids pollutographs compared at
100	OUTLET
101	BANK 54"
	BOD pollutographs compared at
102	OUTLET
103	BANK 54"
	Storm of 8/25-6/71; GP-1; DWF data-1; PCTZER-1; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
104	OUTLET
105	11' x 9'
106	BANK 78"
107	BANK 54"
	Suspended solids pollutographs compared at
108	OUTLET
109	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	BOD pollutographs compared at
110	OUTLET
111	BANK 54"
	Storm of 8/25-6/71; GP-1; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
112	OUTLET
113	11' x 9'
114	BANK 78"
115	BANK 54"
	Suspended solids pollutographs compared at
116	OUTLET
117	BANK 54"
	BOD pollutographs compared at
118	OUTLET
119	BANK 54"
	Storm of 8/25-6/71; GP-2; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
120	OUTLET
121	11' x 9'
122	BANK 78"
123	BANK 54"
	Suspended solids pollutographs compared at
124	OUTLET
125	BANK 54"
	BOD pollutographs compared at
126	OUTLET
127	BANK 54"
	Storm of 4/19/72; GP-0; DWF data-2; PCTZER-2; INFILTRATION data-2; DINFIL-1.
	No actual comparisons have been made here. The field data and computer output, however, have been supplied (see Appendix C).

TABLE A-1 (continued)
b) BANK TESTING

Comparison Number	Description of Input and Comparisons
	Storm of 5/7/71; GP-0, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1. Hydrograph compared at
128	BANK 54"
	Storm of 5/7/71; GP-4, DWF data-4; PCTZER-3; INFILTRATION data-3; DINFIL-2. Hydrograph compared at
129	BANK 54"
	Storm of 5/7/71; GP-4, DWF data-4; PCTZER-3; INFILTRATION data-2; DINFIL-2 Hydrograph compared at
130	BANK 54"
	Storm of 5/7/71; GP-3, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1. Hydrograph compared at
131	BANK 54"
	Storm of 5/13/71; GP-3, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1. Hydrograph compared at
132	BANK 54"
	Storm of 5/13/71; GP-4, DWF data-4; PCTZER-3; INFILTRATION data-2; DINFIL-2. Hydrograph compared at
133	BANK 54"
	Storm of 8/3-4/71; GP-0, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1. Hydrograph compared at
134	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Suspended solids pollutographs compared at
135	BANK 54"
	BOD pollutographs compared at
136	BANK 54"
	Storm of 8/3-4/71; GP-3, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
137	BANK 54"
	Storm of 8/3-4/71; GP-4, DWF data-4; PCTZER-3; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
138	BANK 54"
	Suspended solids pollutographs compared at
139	BANK 54"
	BOD pollutographs compared at
140	BANK 54"
	Storm of 8/3-4/71; GP-0, DWF data-3; PCTZER-4; INFILTRATION data-3; DINFIL-2.
	Hydrographs compared at
141	BANK 54"
	Suspended solids pollutographs compared at
142	BANK 54"
	BOD pollutographs compared at
143	BANK 54"

TABLE A-1 (continued)

Comparison Number	Description of Input and Comparisons
	Storm of 8/3-4/71; GP-0, DWF data-3; PCTZER-4; INFILTRATION data-2; DINFIL-2.
	Hydrographs compared at
144	BANK 54"
	Suspended solids pollutographs compared at
145	BANK 54"
	BOD pollutographs compared at
146	BANK 54"
	Storm of 8/3-4/71; GP-4, DWF data-4; PCTZER-4; INFILTRATION data-2; DINFIL-2.
	Hydrographs compared at
147	BANK 54"
	Suspended solids pollutographs compared at
148	BANK 54"
	BOD pollutographs compared at
149	BANK 54"
	Storm of 12/6/71; GP-3, DWF data-3; PCTZER-3; INFILTRATION data-2; DINFIL-1.
	Hydrographs compared at
150	BANK 54"
	Storm of 12/6/71; GP-4, DWF data-4; PCTZER-3; INFILTRATION data-2; DINFIL-2.
	Hydrograph compared at
151	BANK 54"

SUMMARY AND INDEX FOR GRAPHS COMPARING SWMM RESULTS AND FIELD DATA.

General Notes:

Each comparison curve is represented by a letter-number or letter-letter combination referring to the Graph number on which it is plotted. For example, the curve comparing computed BOD with measured BOD for the storm of November 14, 1970 at the 12' x 9' station are all indicated on Graph number A17. Blank spaces indicate no field data available and hence no graph. In the Bank Watershed chart spaces may also indicate that certain input variations were not tested.

Notes on Bloody Run Index Chart (Table A-2):

Other input data, DWF, PCTZER, and INFILTRATION data were also varied but not to the extent that they affected the output curves significantly. These factors have, therefore, been deleted from the index, although all data variations can be checked in Table A-1.

Notes on Bank Index Chart (Table A-3):

In this chart all of the input variables have been included (with the exception of DWF data which is dependent on the GP factor).

TABLE A-2 SUMMARY AND INDEX FOR GRAPHS COMPARING SWMM RESULTS AND FIELD DATA.

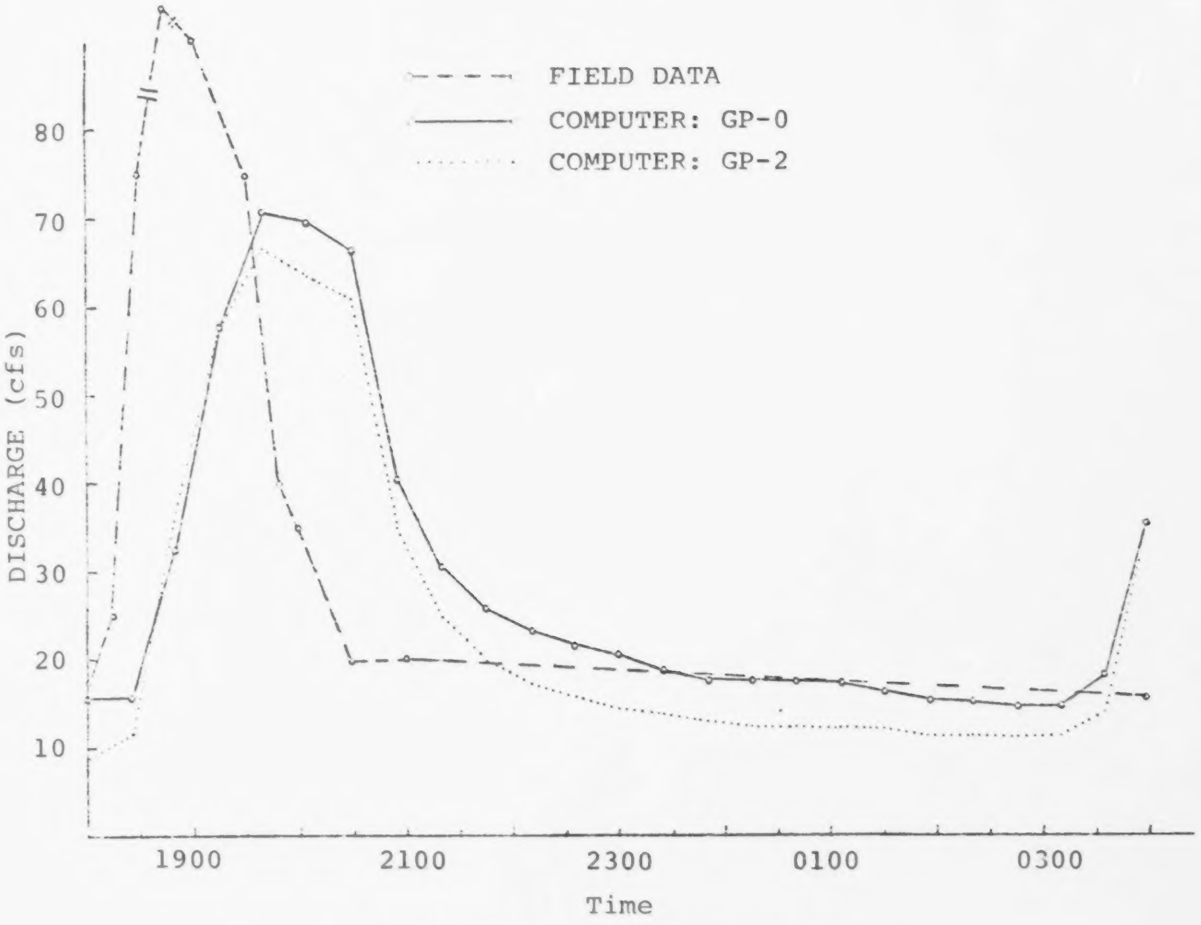
Bloody Run Watershed

		10/20/70			11/9/70			11/14/70			5/7/71			5/13/71			8/25-26/71		
		Q	SS	BOD	Q	SS	BOD	Q	SS	BOD	Q	SS	BOD	Q	SS	BOD	Q	SS	BOD
OUTLET	GP-0	A43	A44	A46	A01			A11			A21	A25	A27	A29	A33	A34	A35	A39	A41
	GP-1							A11									A35	A39	A41
	GP-2				A01			A11			A21	A25	A27	A29	A33	A34	A35	A39	A41
12 x 9	GP-0				A02		A07	A12		A17	A22	A26	A28	A30					
	GP-1							A12		A17									
	GP-2				A02		A07	A12		A17	A22	A26	A28	A30					
11 x 9	GP-0				A03		A08	A13		A18	A23			A31			A36		
	GP-1							A13		A18							A36		
	GP-2				A03		A08	A13		A18	A23			A31			A36		
Bank 78	GP-0				A04	A05	A09	A14	A15	A19	A24			A32			A37		
	GP-1							A14	A15	A19							A37		
	GP-2				A04	A05	A09	A14	A15	A19	A24			A32			A37		
Bank 54	GP-0		A45	A47		A06	A10		A16	A20	BA			BB			A38	A40	A42
	GP-1								A16	A20							A38	A40	A42
	GP-2					A06	A10		A16	A20	BA			BB			A38	A40	A42

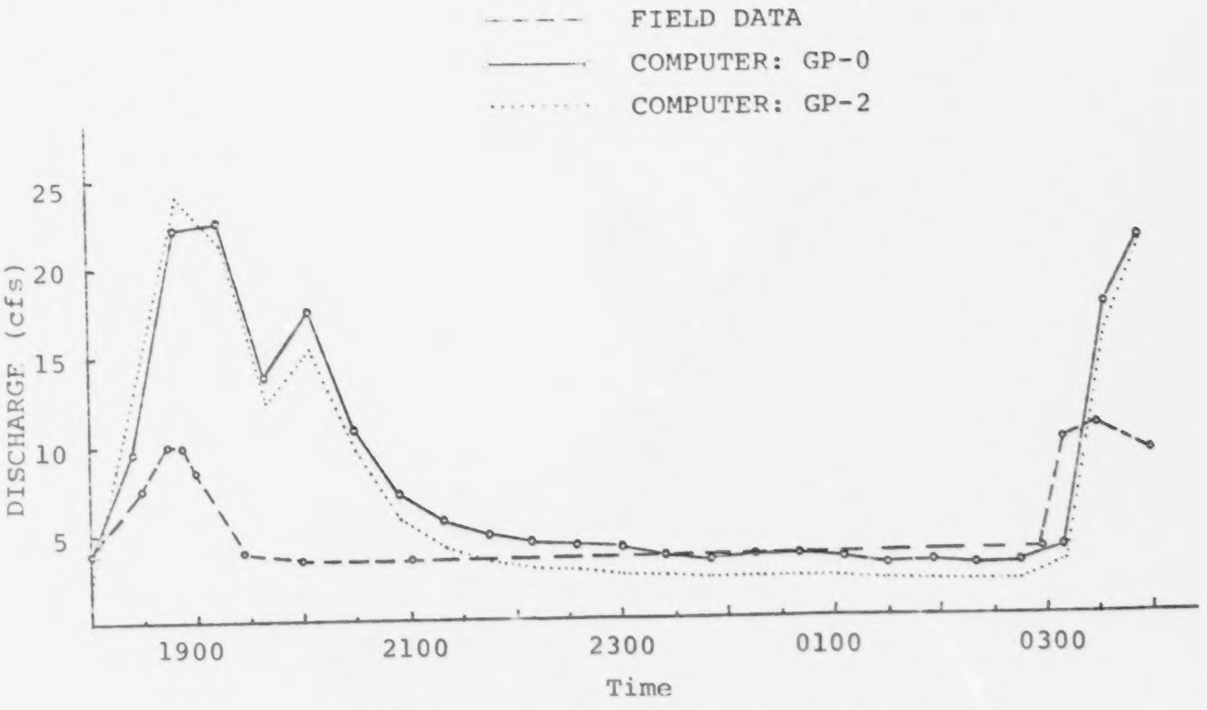
TABLE A-3 SUMMARY AND INDEX FOR GRAPHS COMPARING SWMM RESULTS AND FIELD DATA.

Bank Watershed

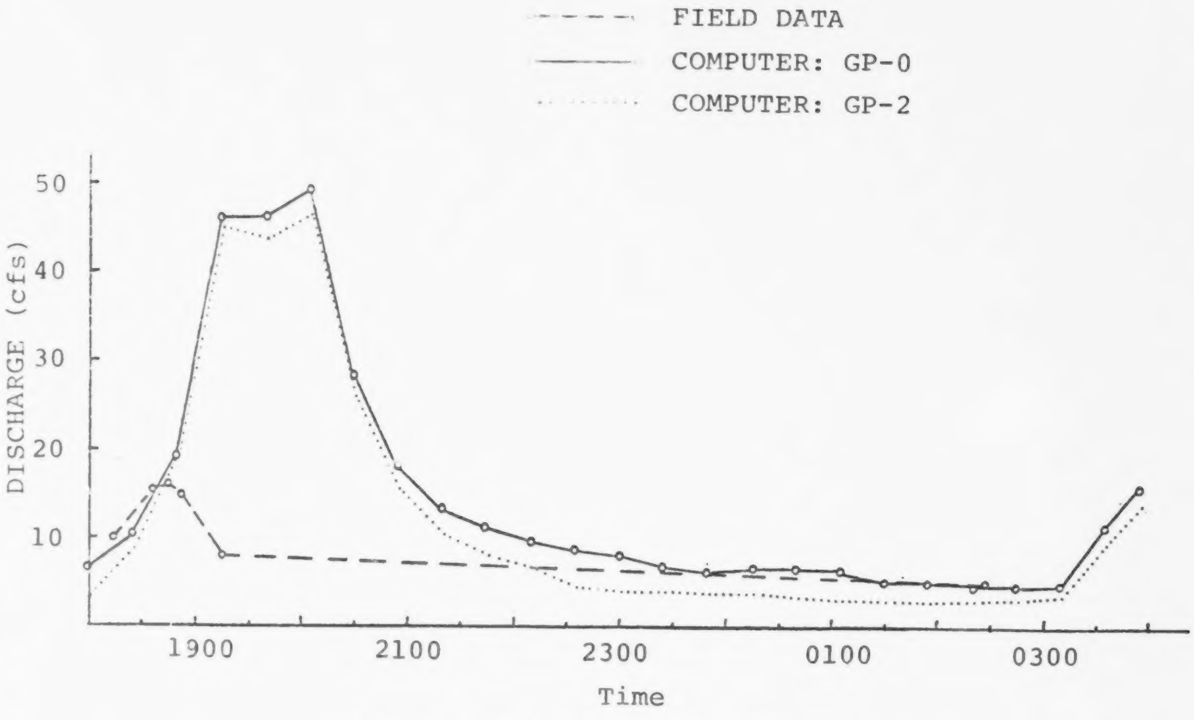
			5/7/71						5/13/71						8/3-4/71						12/6/71					
			Q		SS		BOD		Q		SS		BOD		Q		SS		BOD		Q		SS		BOD	
			INFILTRATION data						INFILTRATION data						INFILTRATION data						INFILTRATION data					
			2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3
GP-0	PCTZER-3	DINFIL 1	BA												BD		BG		BK							
		DINFIL 2																								
	PCTZER-4	DINFIL 1																								
		DINFIL 2													BD BF		BJ		BN							
GP-3	PCTZER-3	DINFIL 1	BA						BC						BD						BO					
		DINFIL 2																								
	PCTZER-4	DINFIL 1																								
		DINFIL 2																								
GP-4	PCTZER-3	DINFIL 1													BD BE		BG BH BI BJ		BK BL BM BN							
		DINFIL 2	BA	BA					BC												BO					
	PCTZER-4	DINFIL 1																								
		DINFIL 2													BD BE BF		BH BI BJ		BL BM BN							



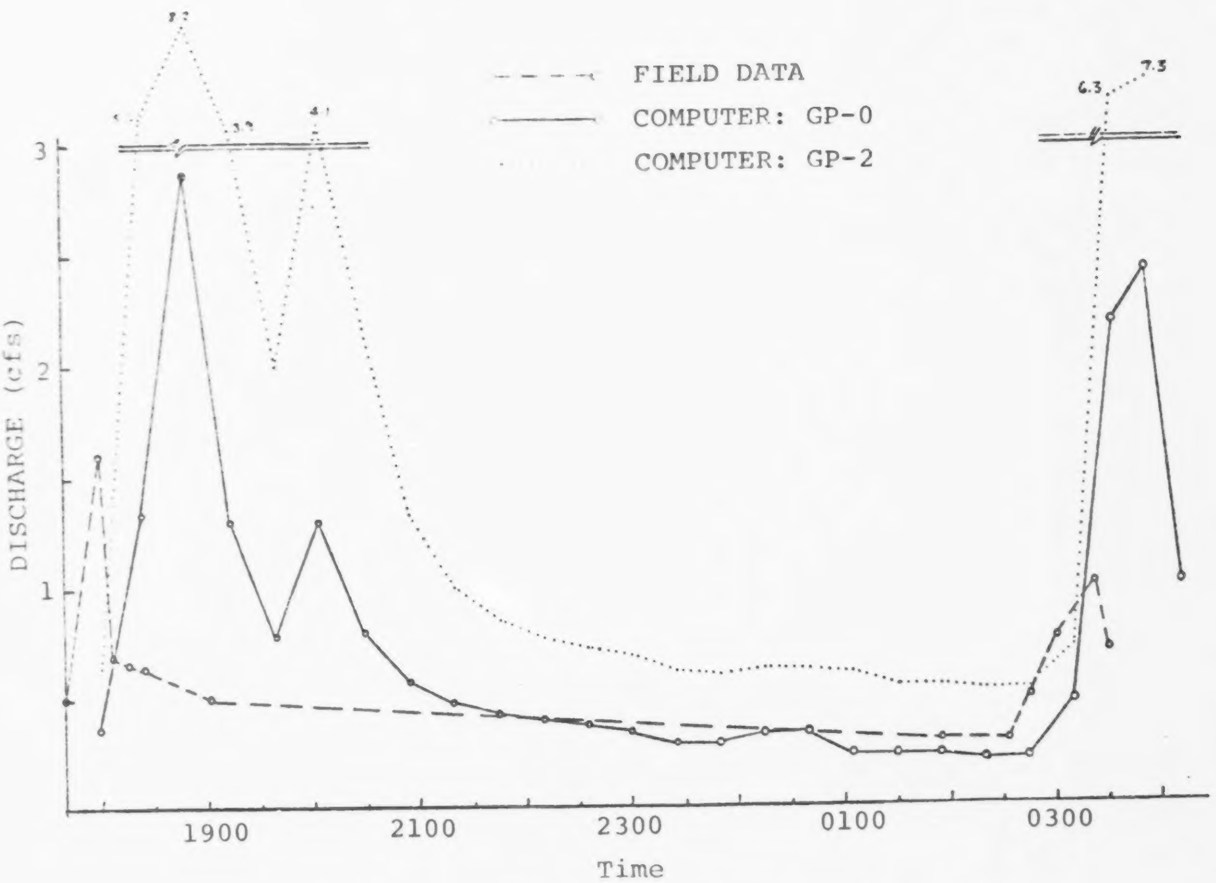
Graph A-01. DISCHARGE VS. TIME
OUTLET MONITORING STATION
NOV. 9, 1970



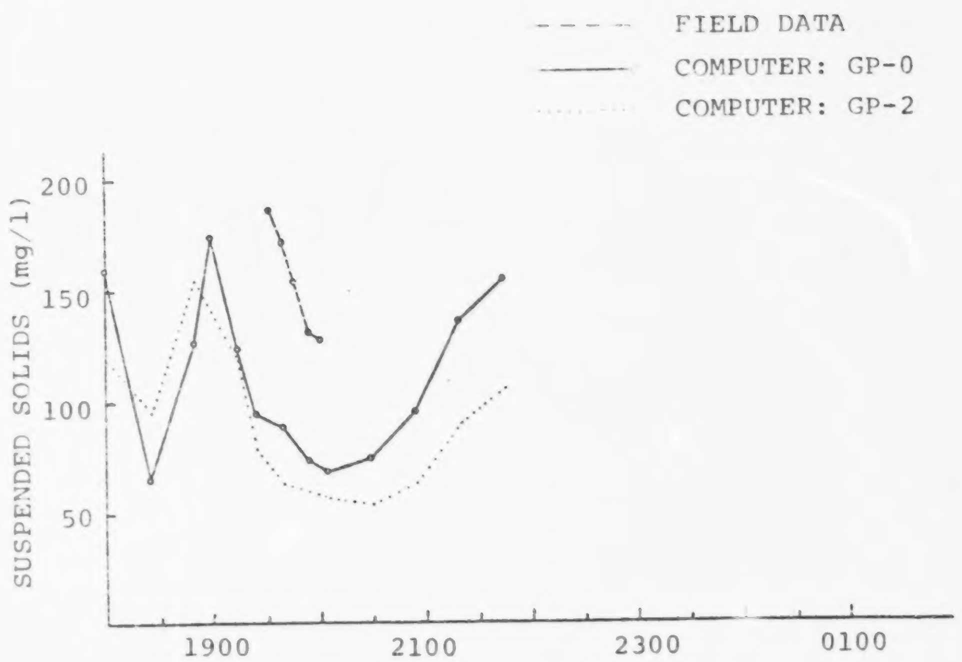
Graph A-02. DISCHARGE VS. TIME
LONGVIEW 12'x9' MONITORING STATION
NOV. 9, 1970



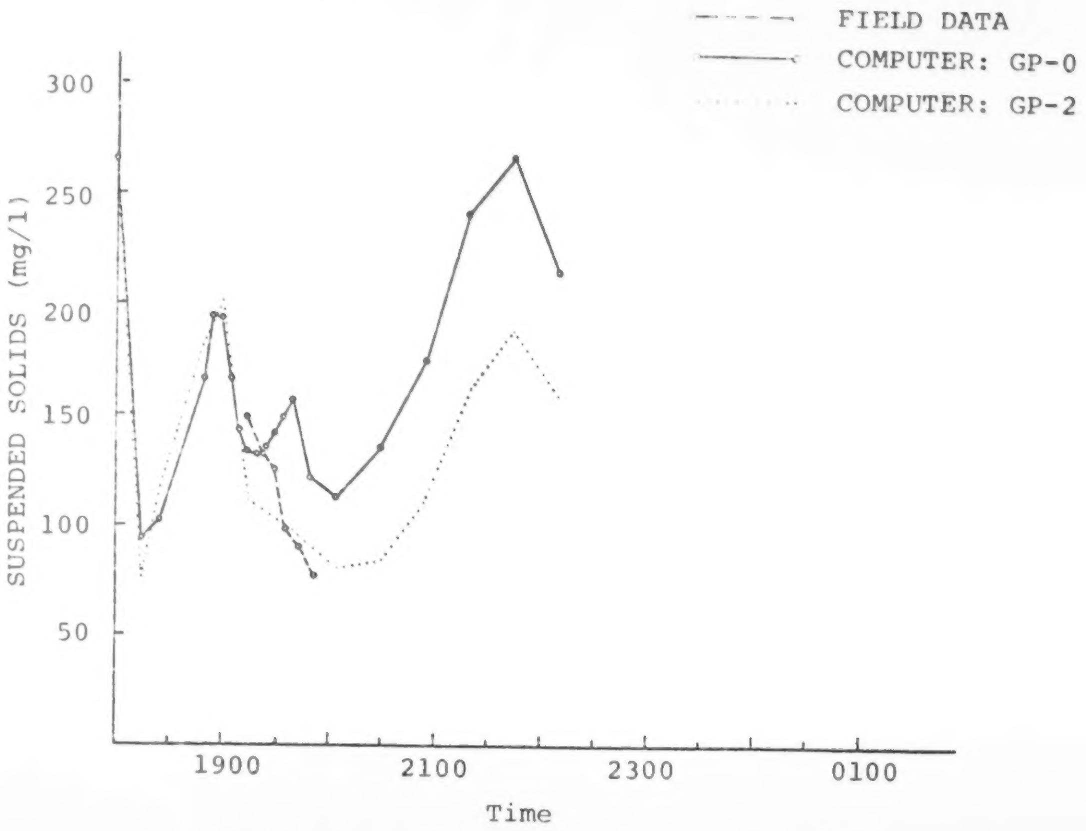
Graph A-03. DISCHARGE VS. TIME
LONGVIEW 11'x9' MONITORING STATION
NOV. 9, 1970



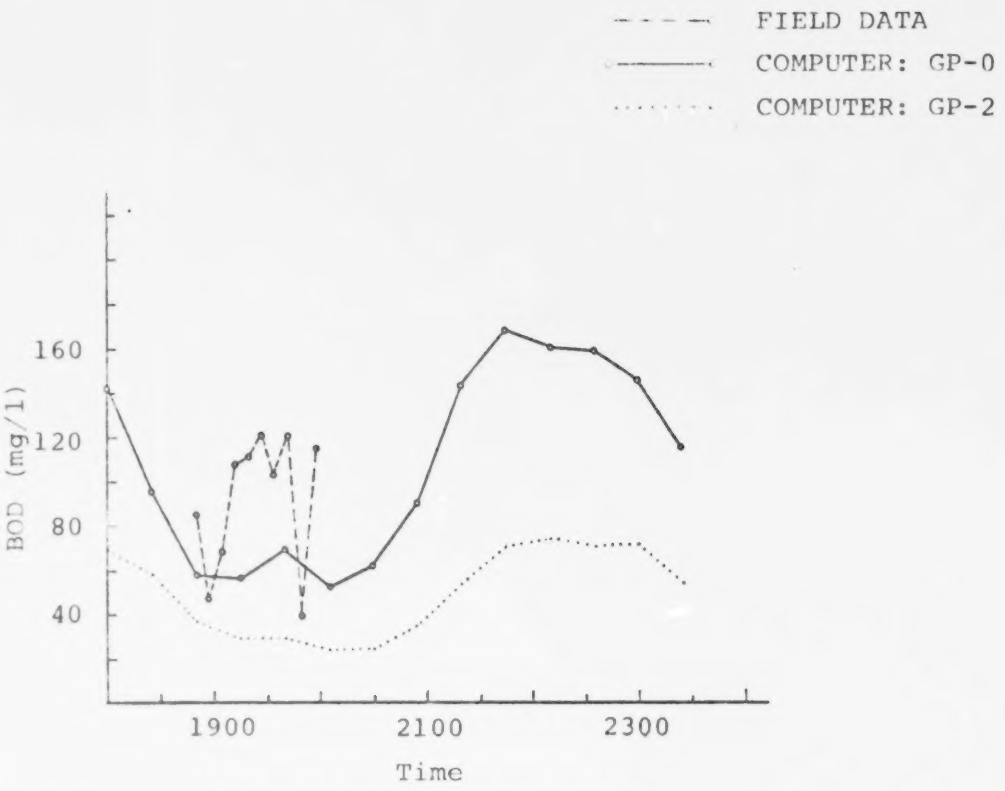
Graph A-04. DISCHARGE VS. TIME
BANK 78" MONITORING STATION
NOV. 9, 1970



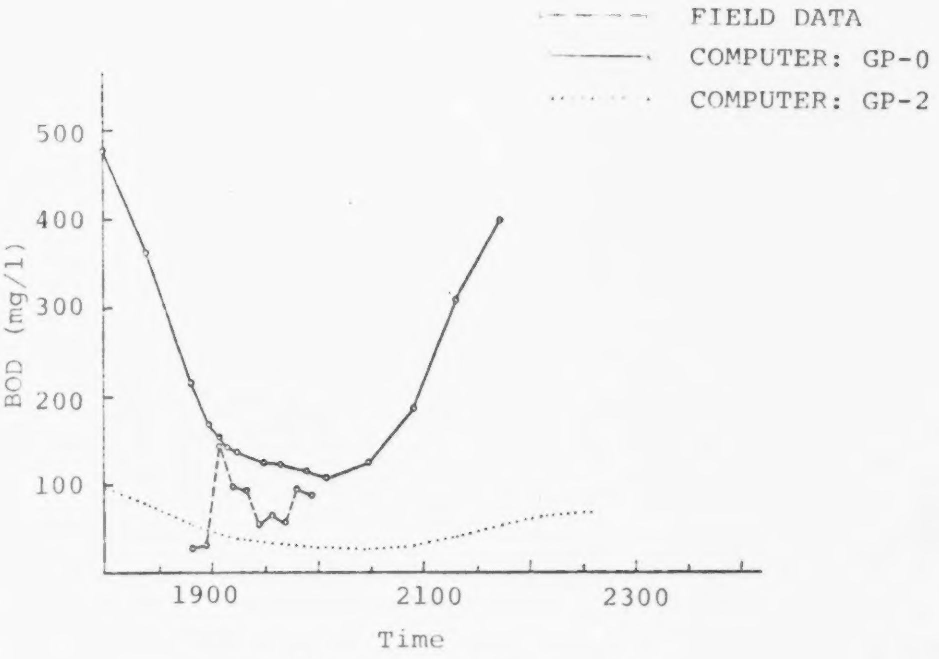
Graph A-65. SUSPENDED SOLIDS VS. TIME
BANK 78" MONITORING STATION
NOV. 9, 1970



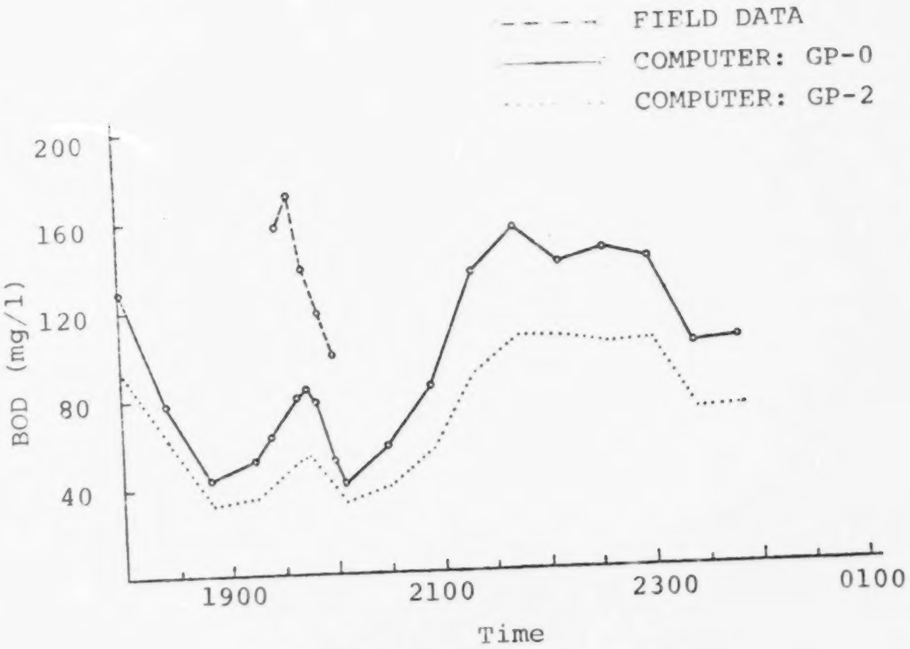
Graph A-06. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
NOV. 9, 1970



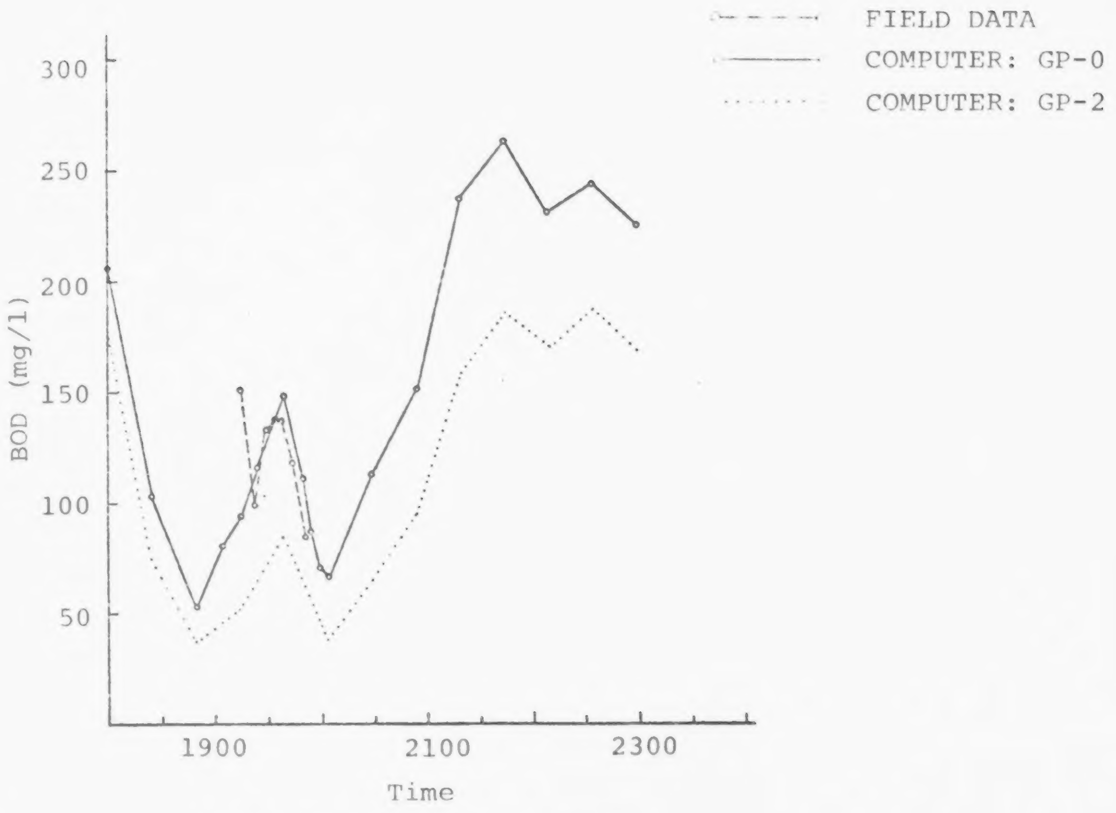
Graph A-07. BOD VS. TIME
LONGVIEW 12'x9' MONITORING STATION
NOV. 9, 1970



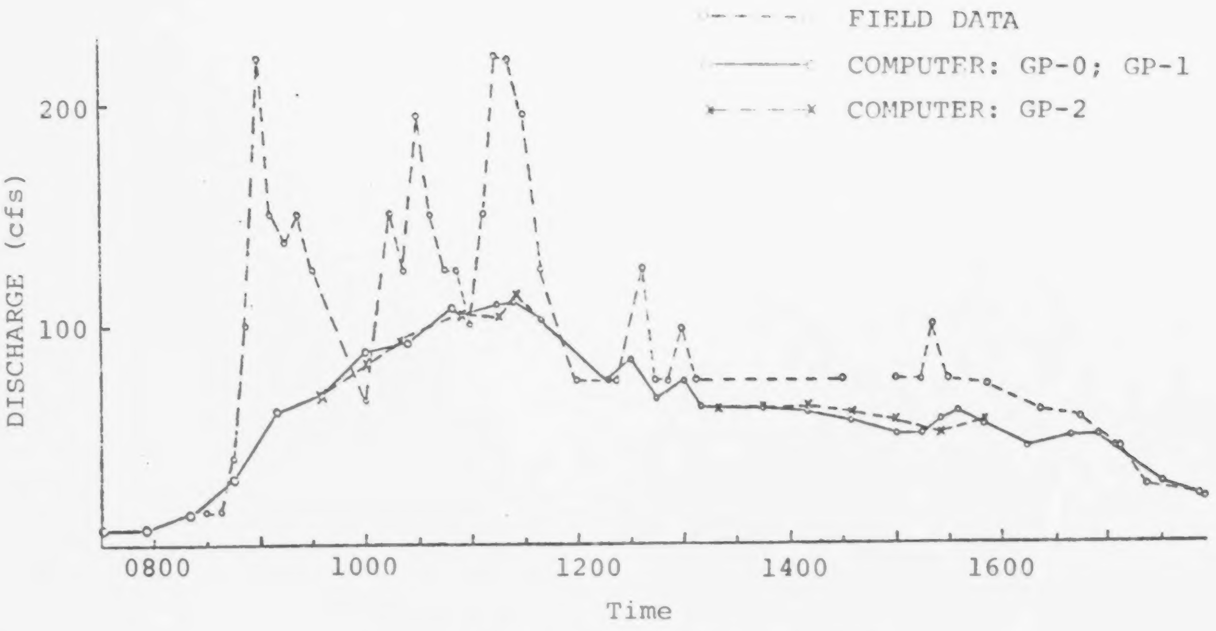
Graph A-08. BOD VS. TIME
LONGVIEW 11'x9' MONITORING STATION
NOV. 9, 1970



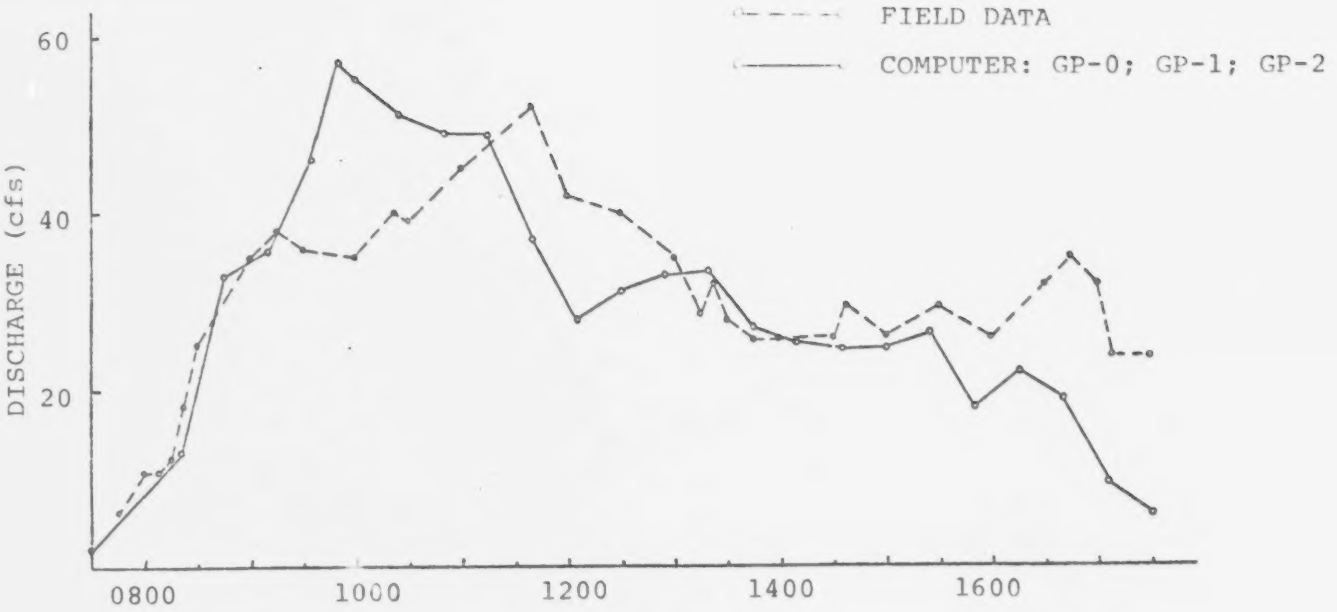
Graph A-09. BOD VS. TIME
BANK 78" MONITORING STATION
NOV. 9, 1970



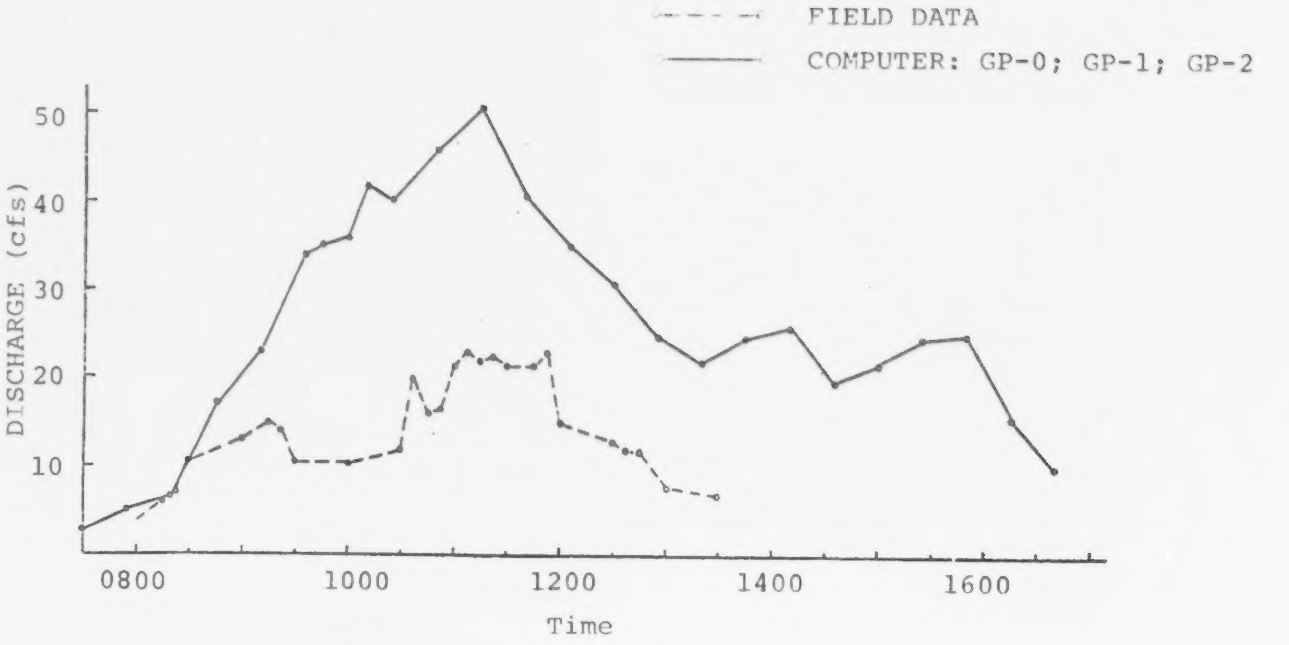
Graph A-10. BOD VS. TIME
BANK 54" MONITORING STATION
NOV. 9, 1970



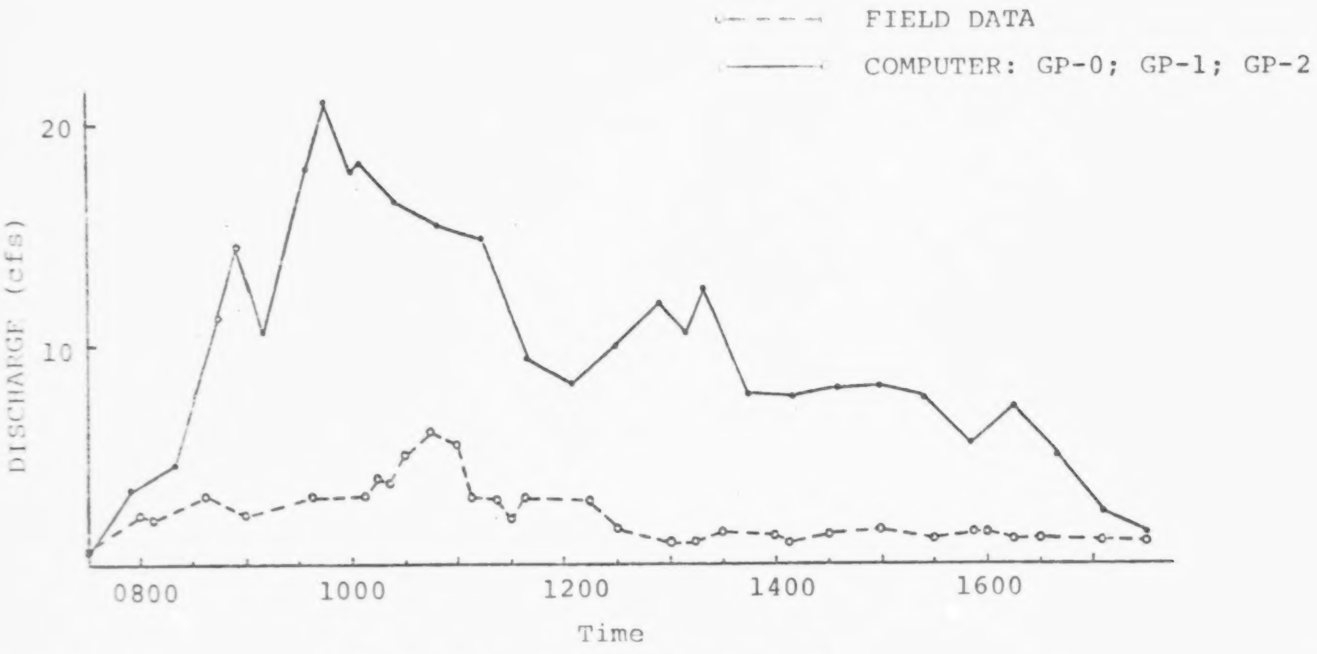
Graph A-11. DISCHARGE VS. TIME
OUTLET MONITORING STATION
NOV. 14, 1970



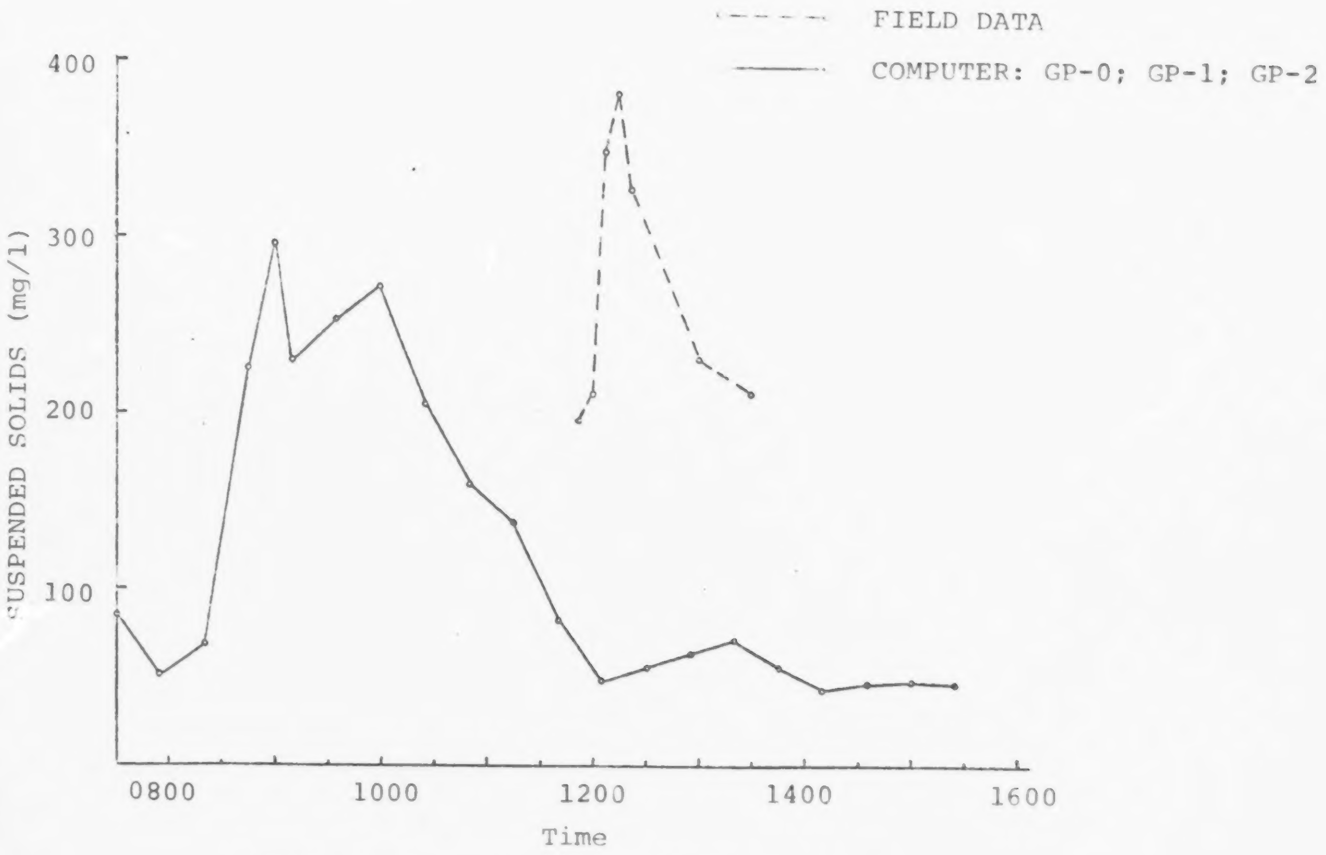
Graph A-12. DISCHARGE VS. TIME
LONGVIEW 12'x9' MONITORING STATION
NOV. 14, 1970



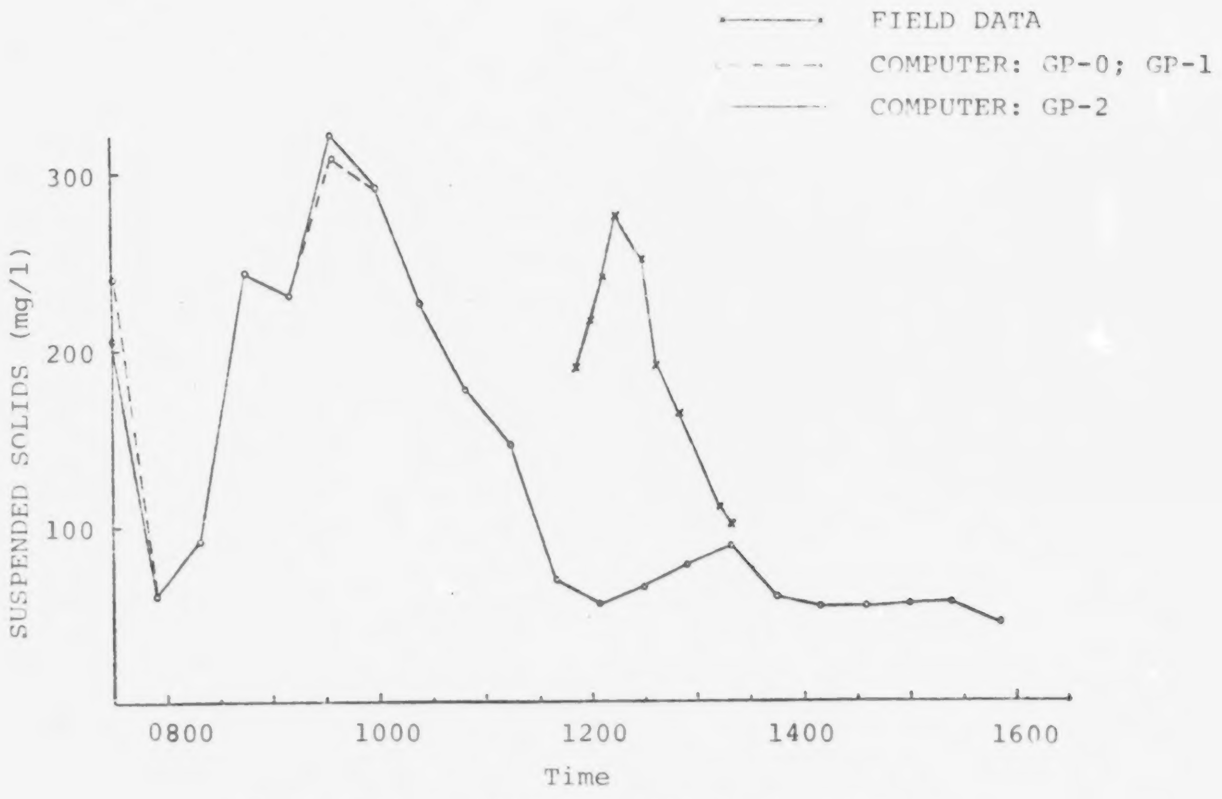
Graph A-13. DISCHARGE VS. TIME
LONGVIEW 11'x9' MONITORING STATION
NOV. 14, 1970



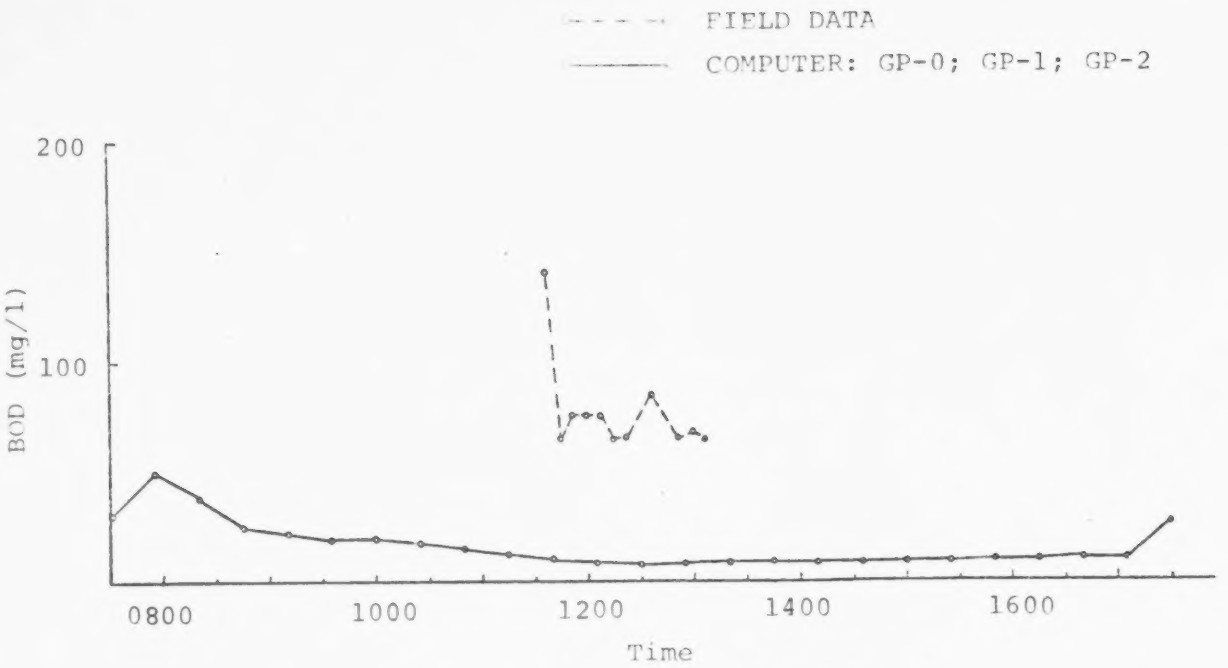
Graph A-14. DISCHARGE VS. TIME
BANK 78" MONITORING STATION
NOV. 14, 1970



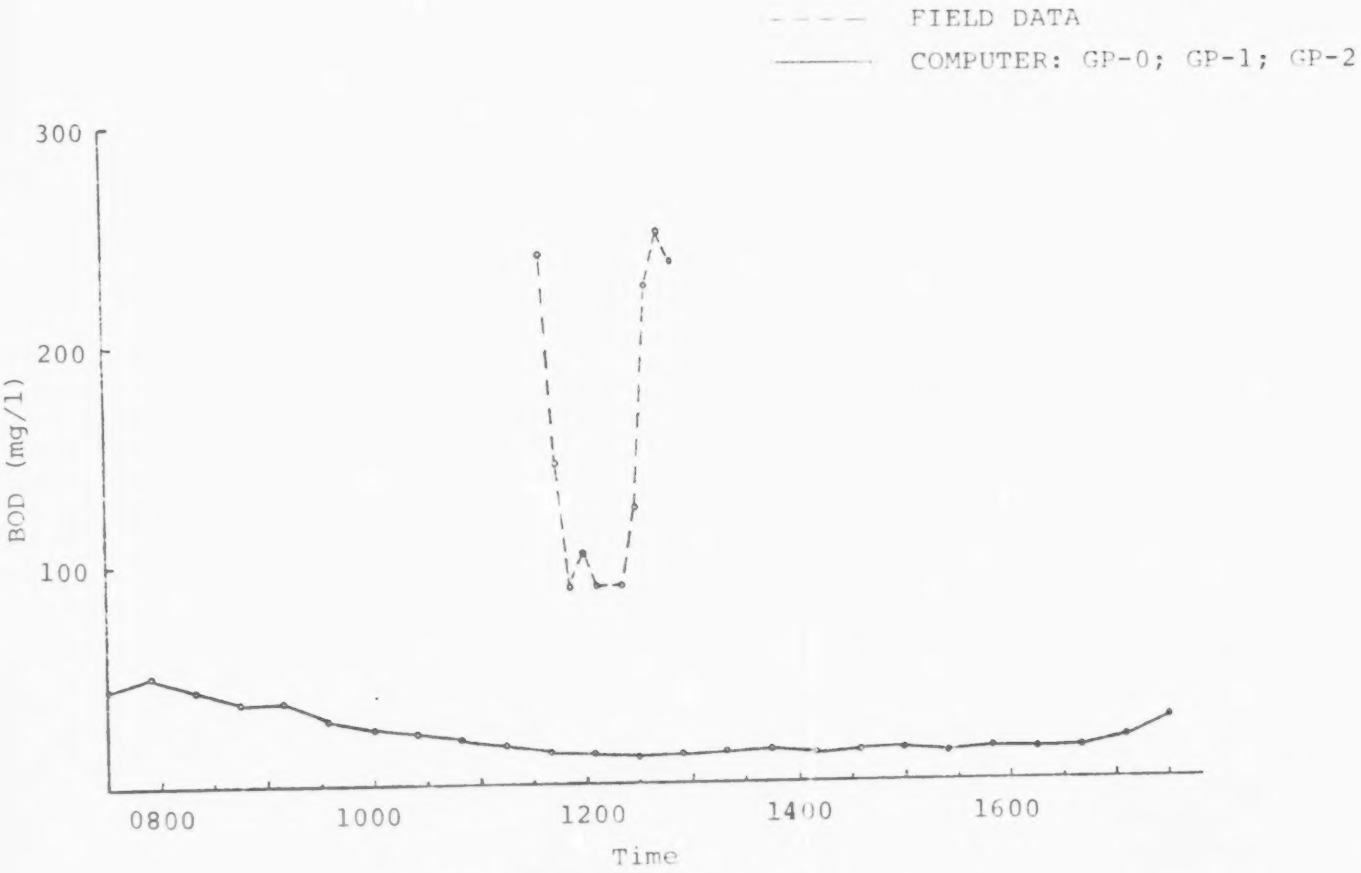
Graph A-15. SUSPENDED SOLIDS VS. TIME
BANK 78" MONITORING STATION
NOV. 14, 1970



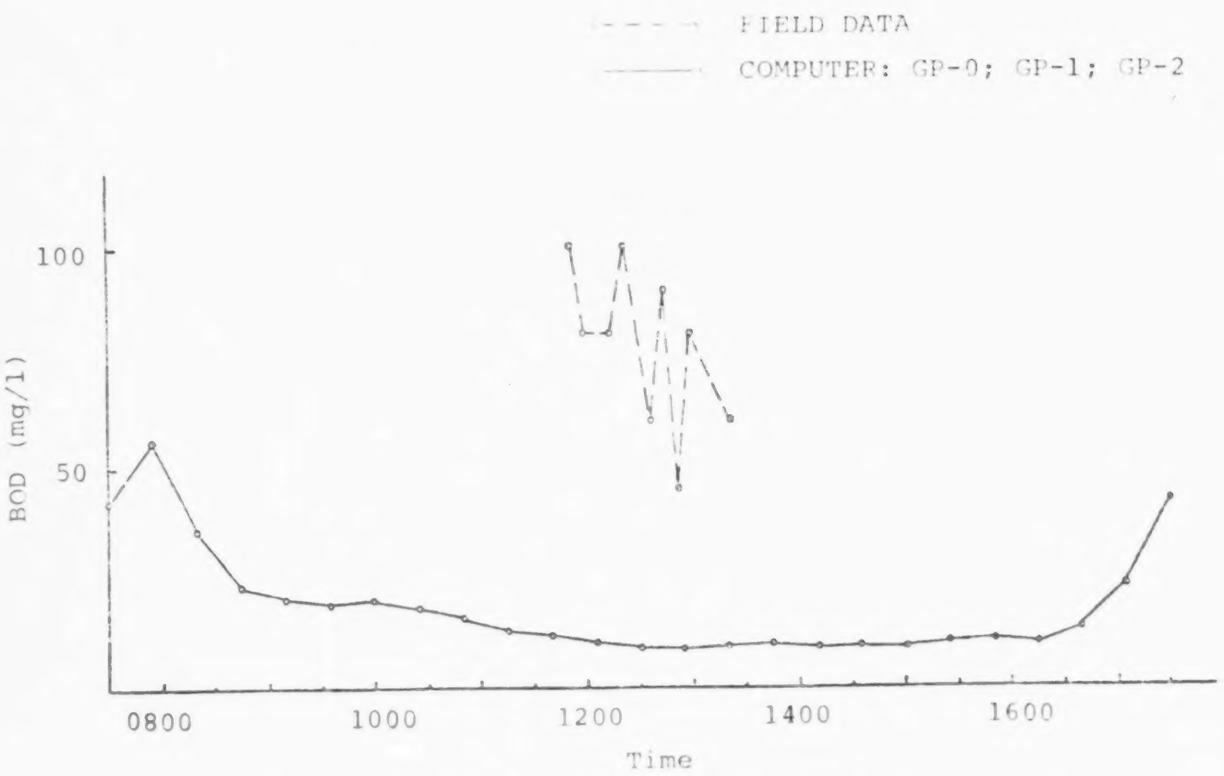
Graph A-16. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
NOV. 14, 1970



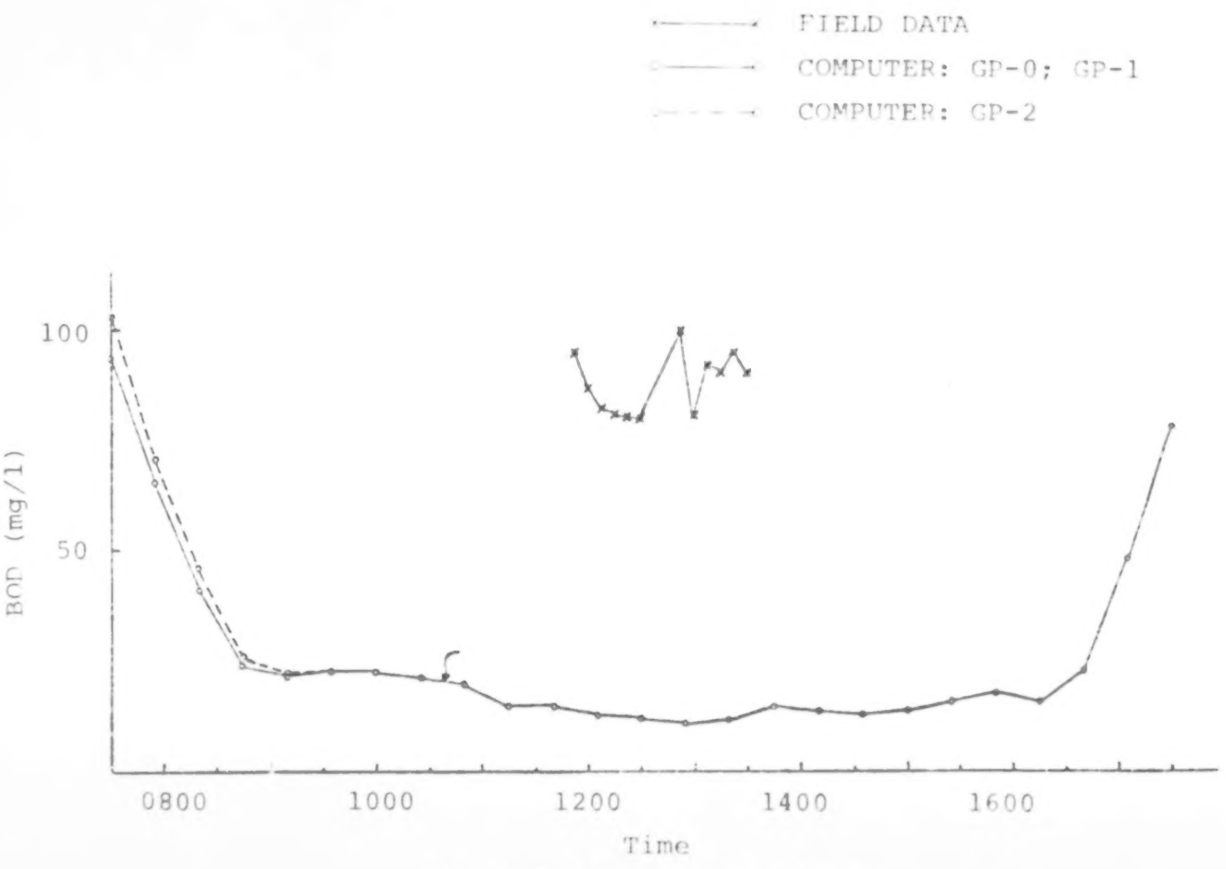
Graph A-17. BOD VS. TIME
LONGVIEW 12'x9' MONITORING STATION
NOV. 14, 1970



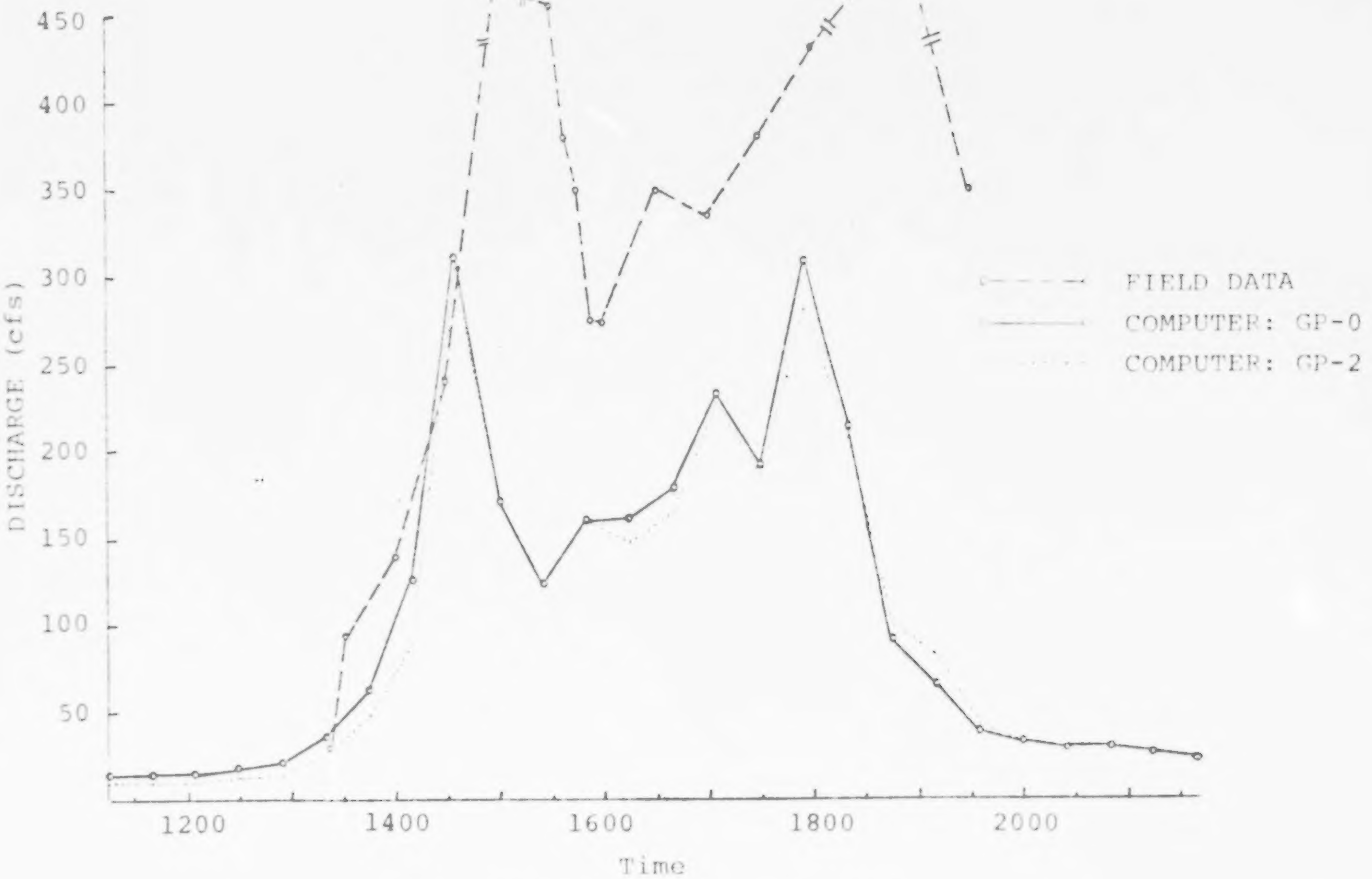
Graph A-18. BOD VS. TIME
LONGVIEW 11'x9' MONITORING STATION
NOV. 14, 1970



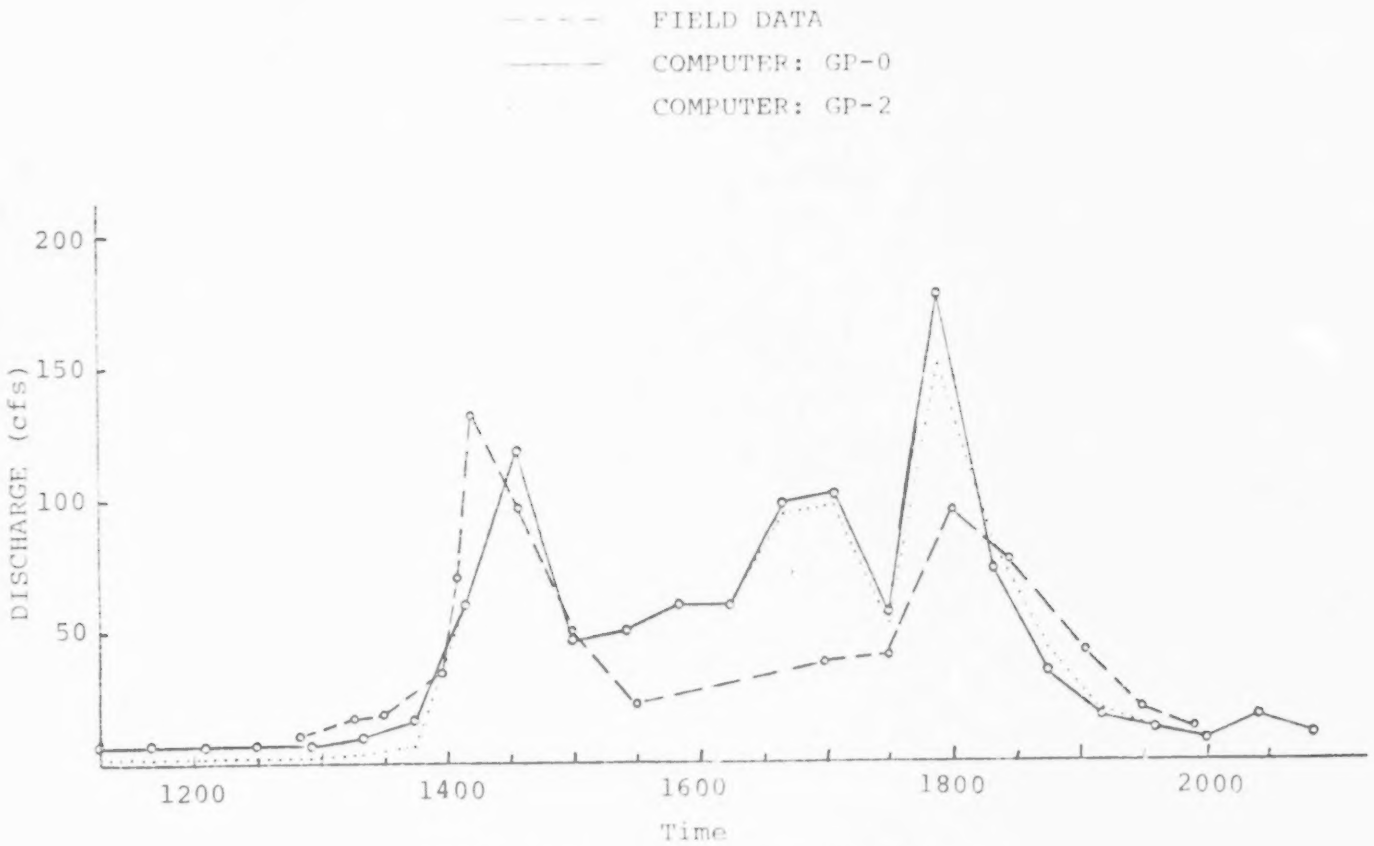
Graph A-19. BOD VS. TIME
BANK 78" MONITORING STATION
NOV. 14, 1970



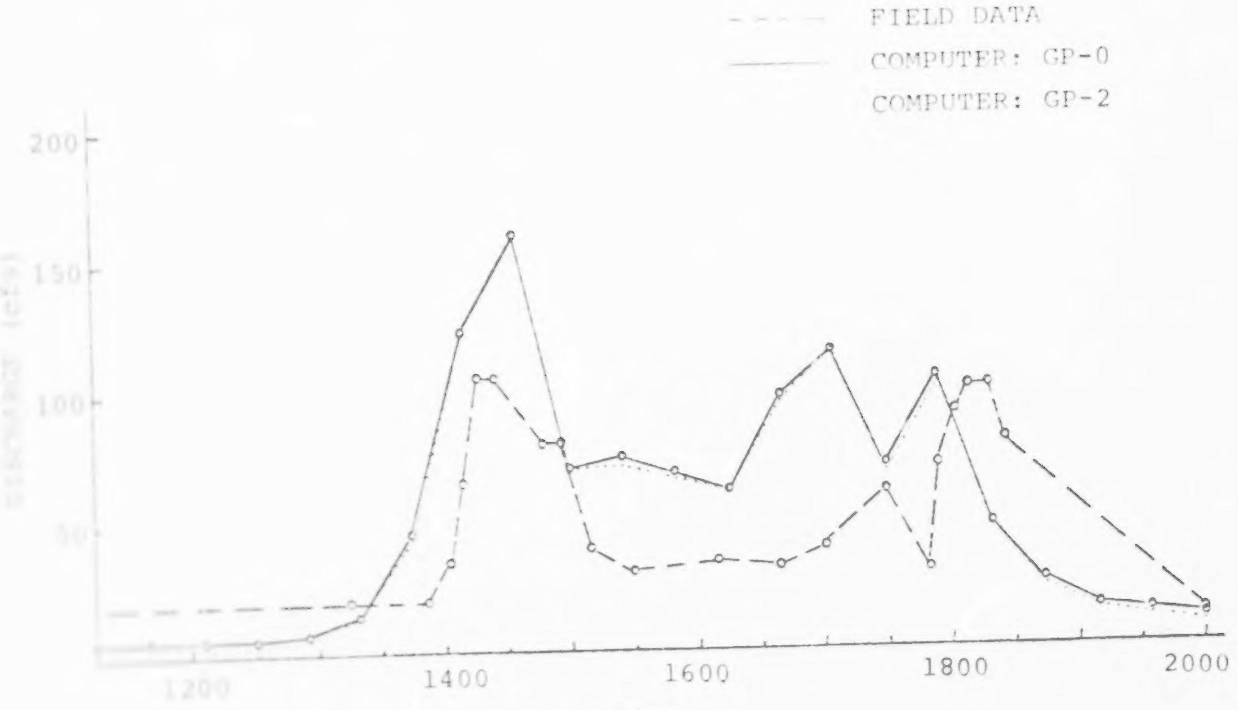
Graph A-20. BOD VS. TIME
BANK 54" MONITORING STATION
NOV. 14, 1970



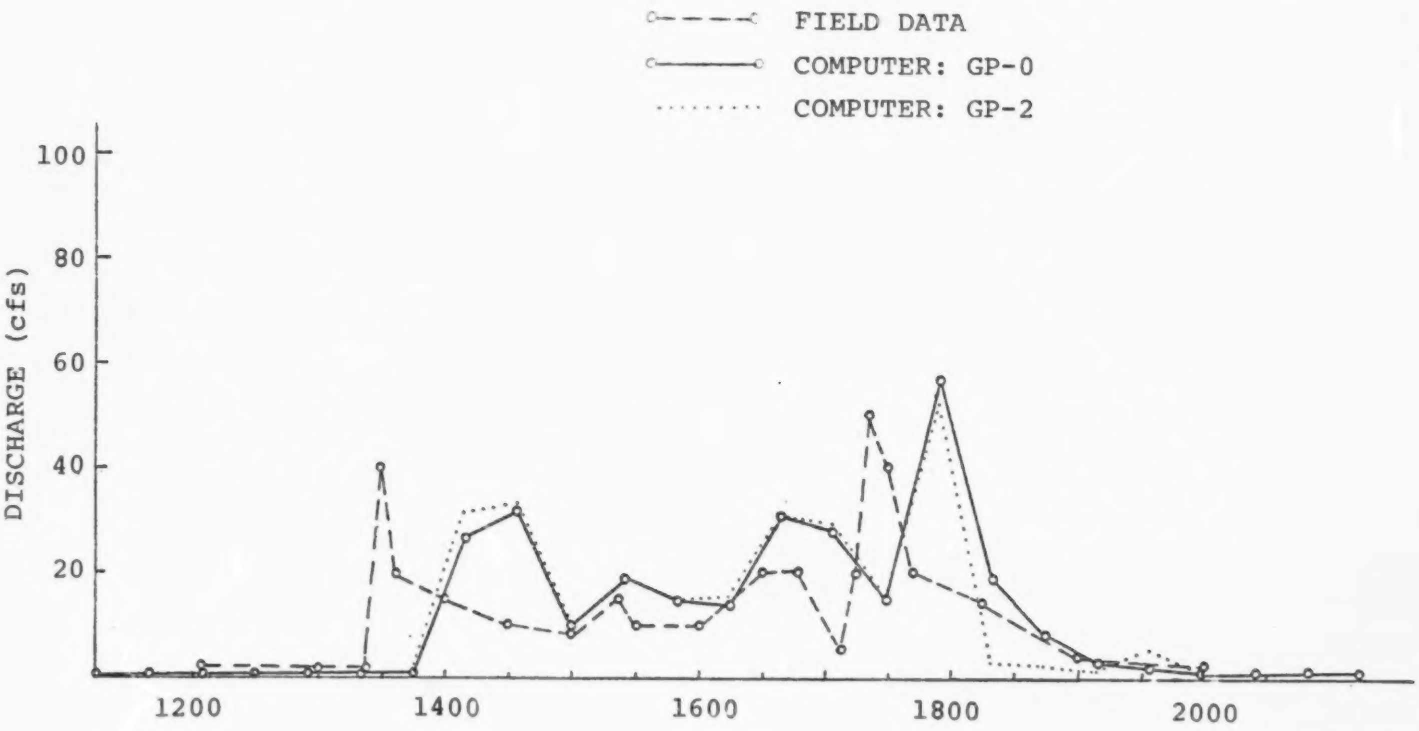
Graph A-21. DISCHARGE VS. TIME
OUTLET MONITORING STATION
MAY 7, 1971



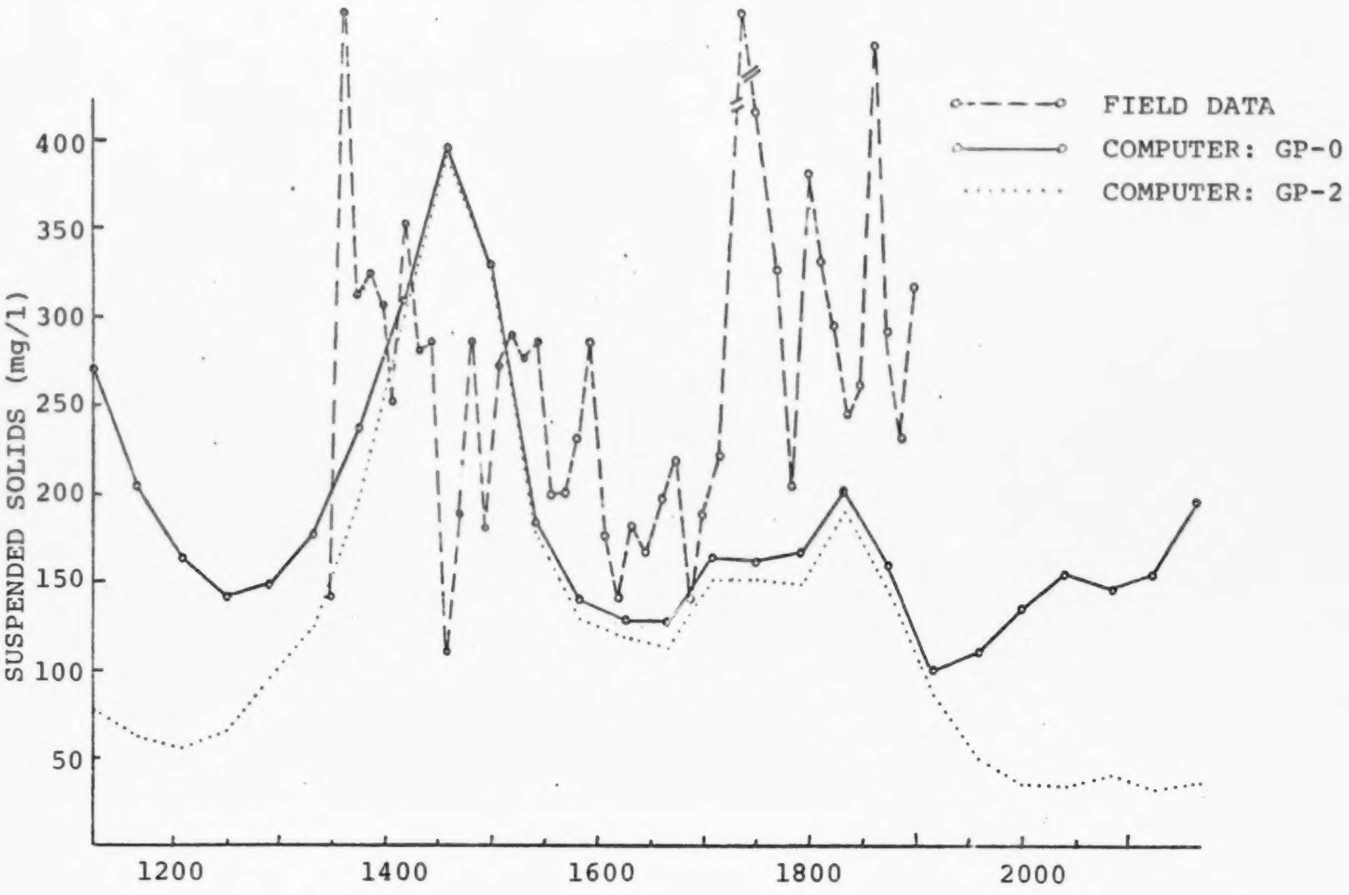
Graph A-22. DISCHARGE VS. TIME
LONGVIEW 12'x9' MONITORING STATION
MAY 7, 1971



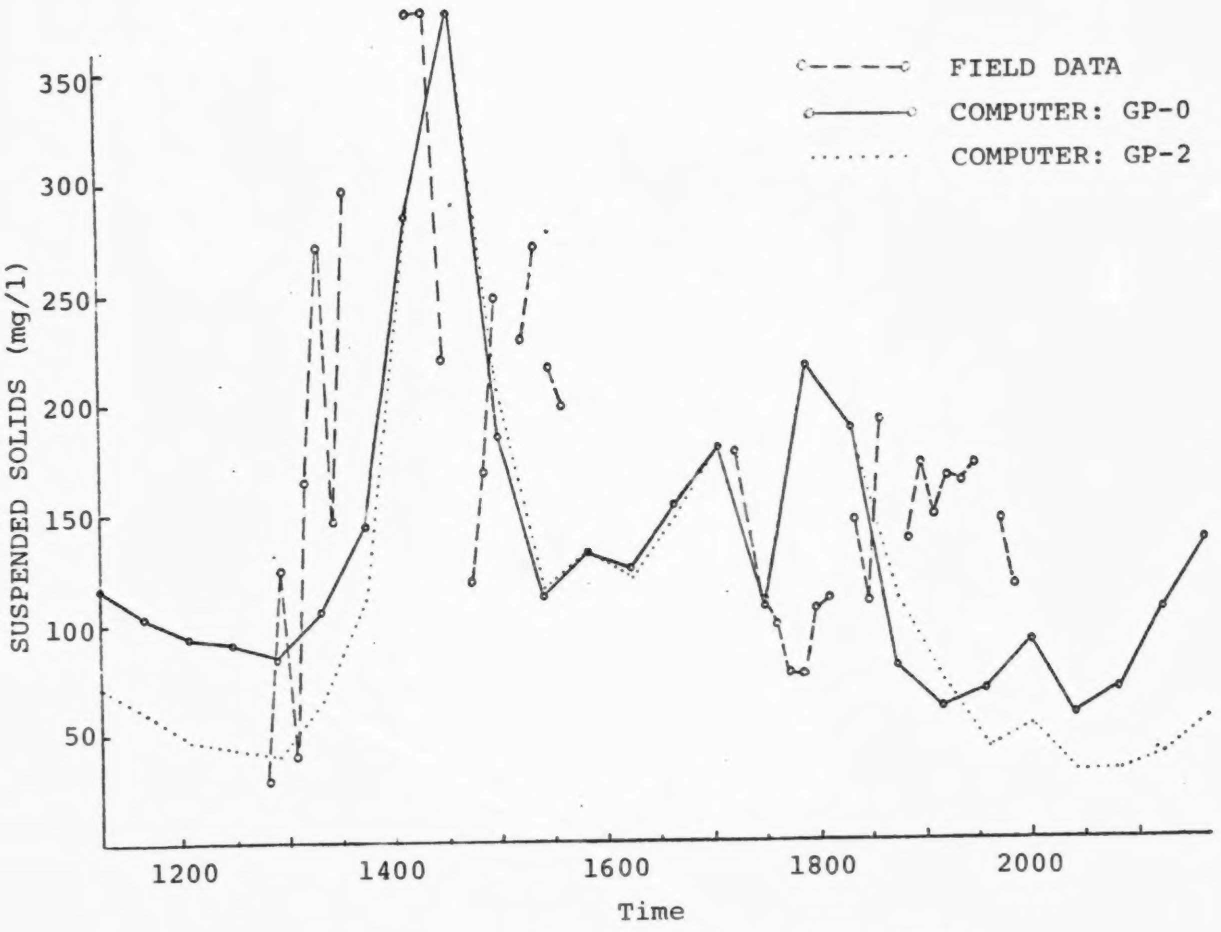
Graph A-23. DISCHARGE VS. TIME
LONGVIEW 11'x9' MONITORING STATION
MAY 7, 1971



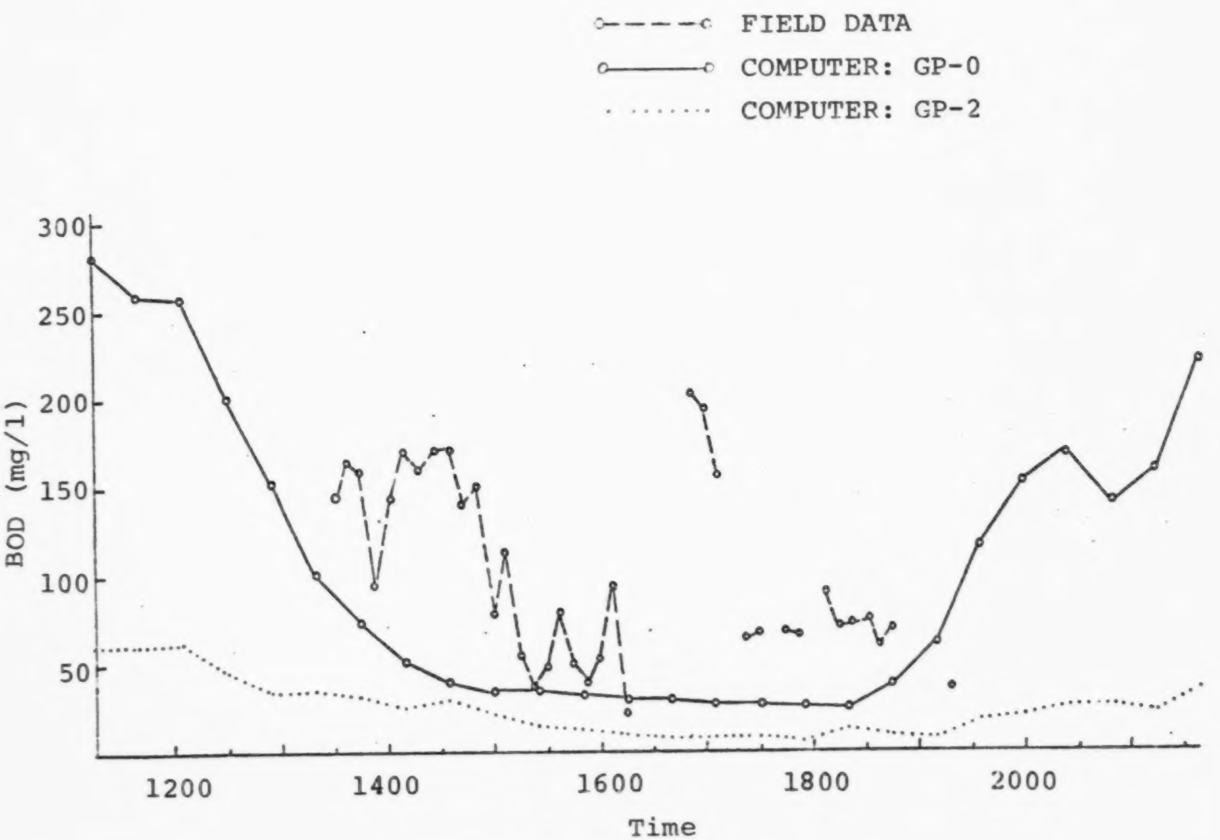
Graph A-24. DISCHARGE VS. TIME
BANK 78" MONITORING STATION
MAY 7, 1971



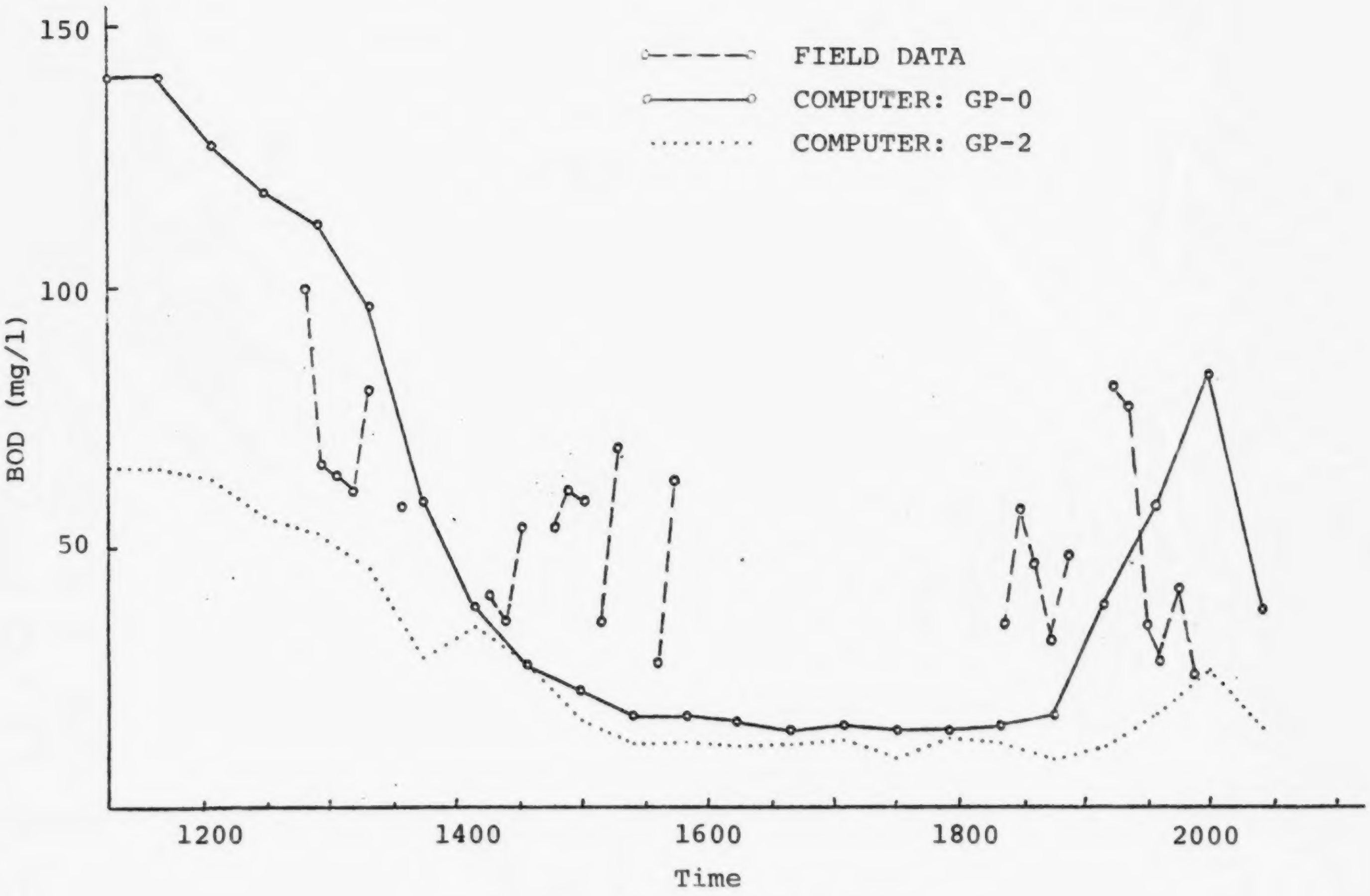
Graph A-25. SUSPENDED SOLIDS VS. TIME
OUTLET MONITORING STATION
MAY 7, 1971



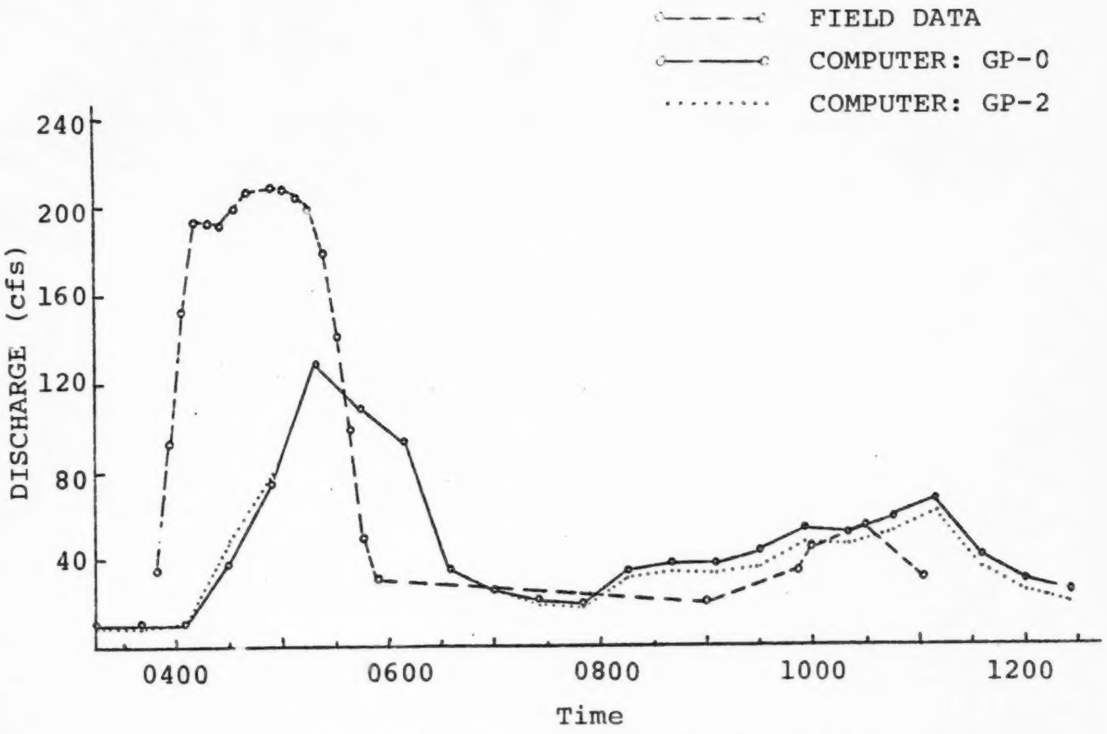
Graph A-26. SUSPENDED SOLIDS VS. TIME
LONGVIEW 12'x9' MONITORING STATION
MAY 7, 1971



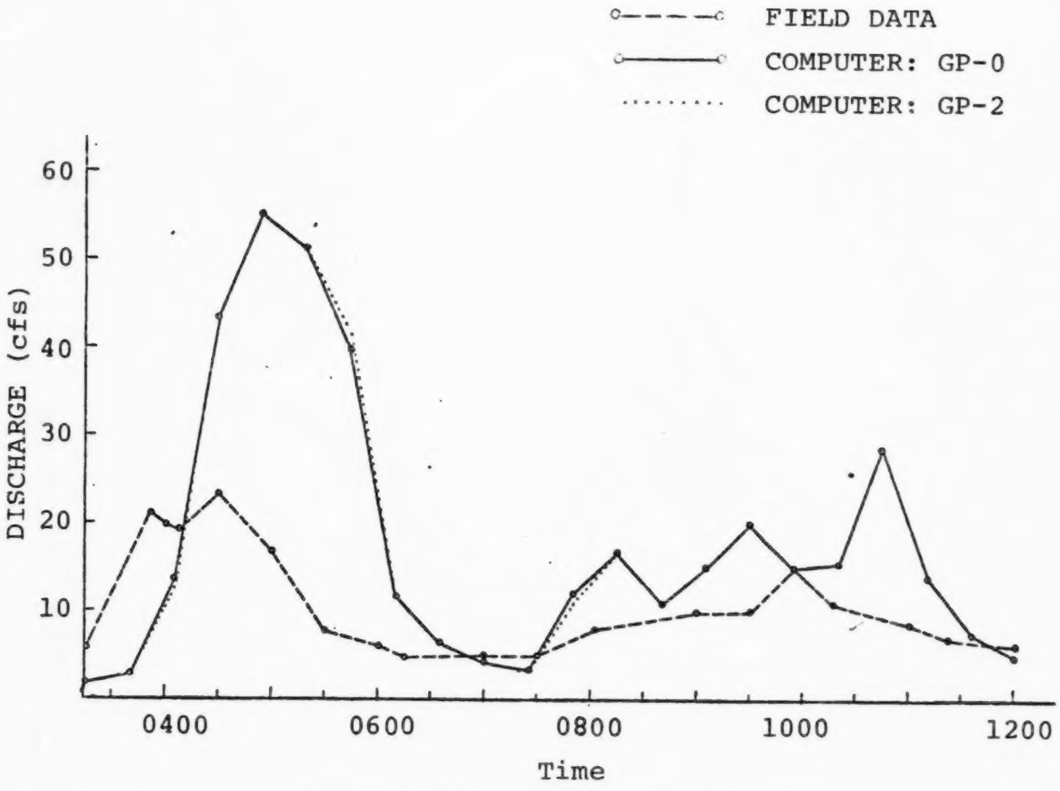
Graph A-27. BOD VS. TIME
OUTLET MONITORING STATION
MAY 7, 1971



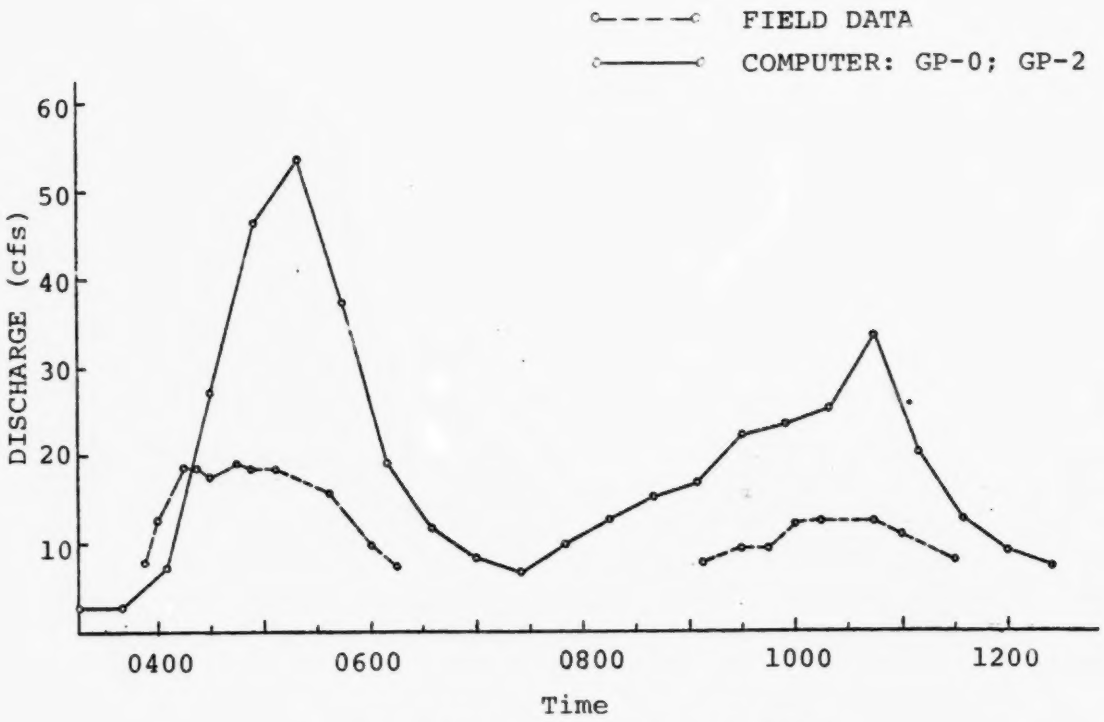
Graph A-28. BOD VS. TIME
LONGVIEW 12'x9' MONITORING STATION
MAY 7, 1971



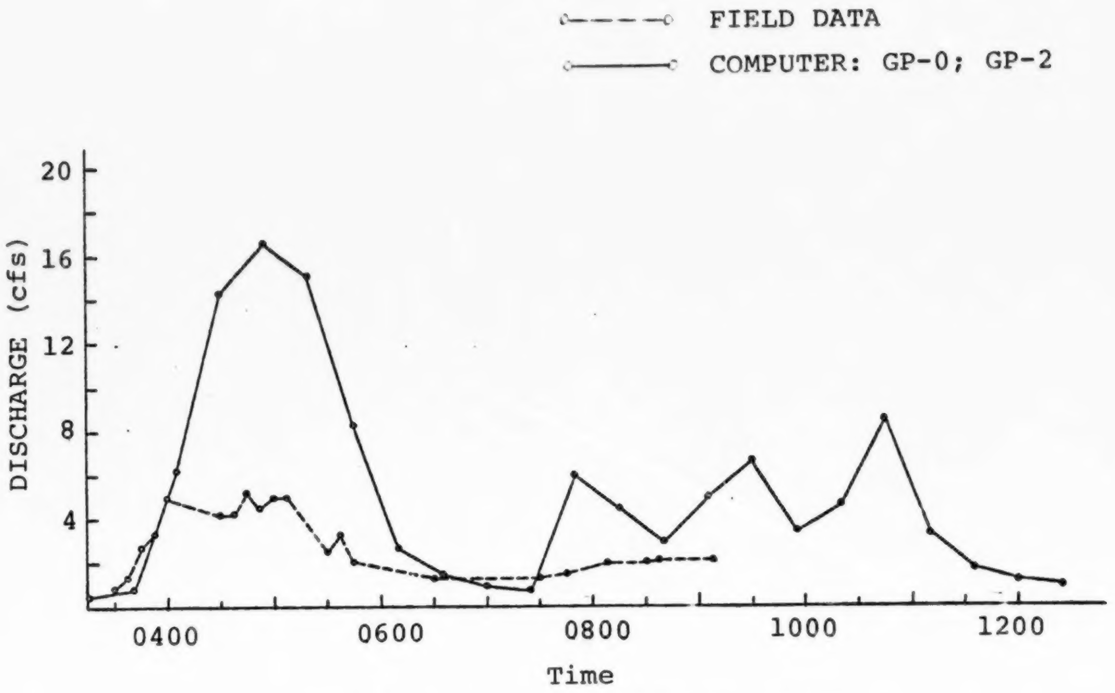
Graph A-29. DISCHARGE VS. TIME
OUTLET MONITORING STATION
MAY 13, 1971



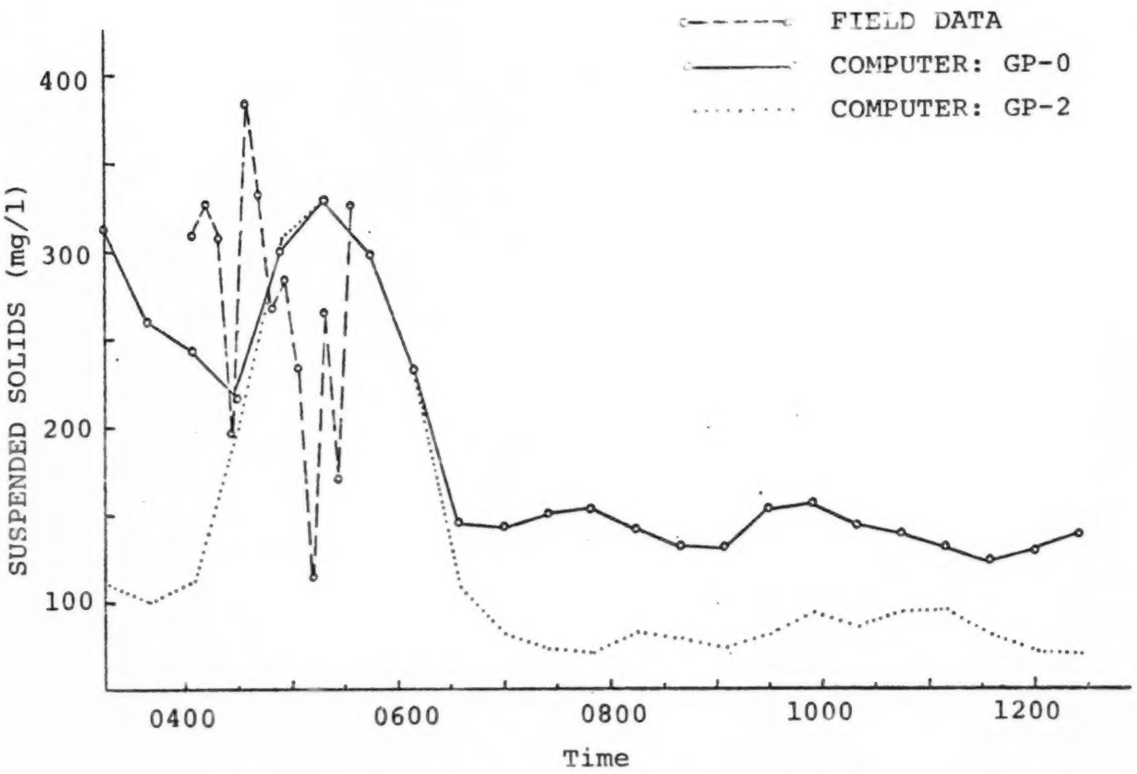
Graph A-30. DISCHARGE VS. TIME
LONGVIEW 12'x9' MONITORING STATION
MAY 13, 1971



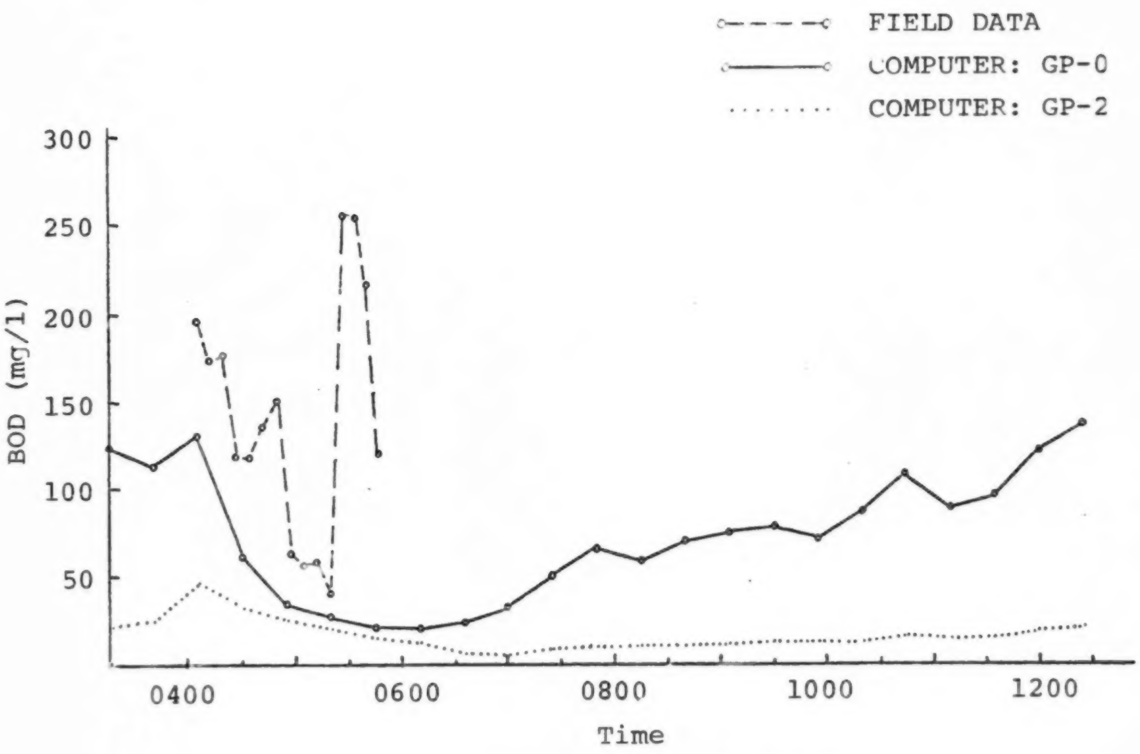
Graph A-31. DISCHARGE VS. TIME
LONGVIEW 11'x9' MONITORING STATION
MAY 13, 1971



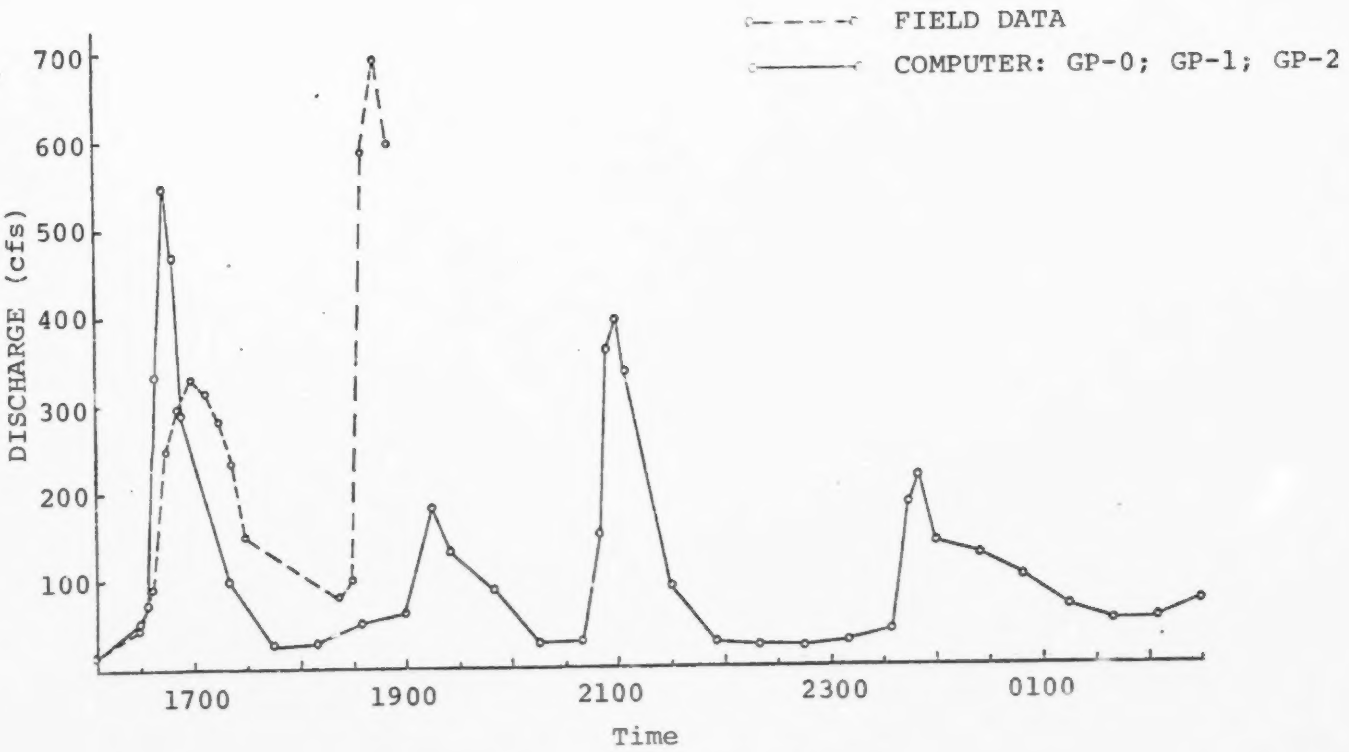
Graph A-32. DISCHARGE VS. TIME
BANK 78" MONITORING STATION
MAY 13, 1971



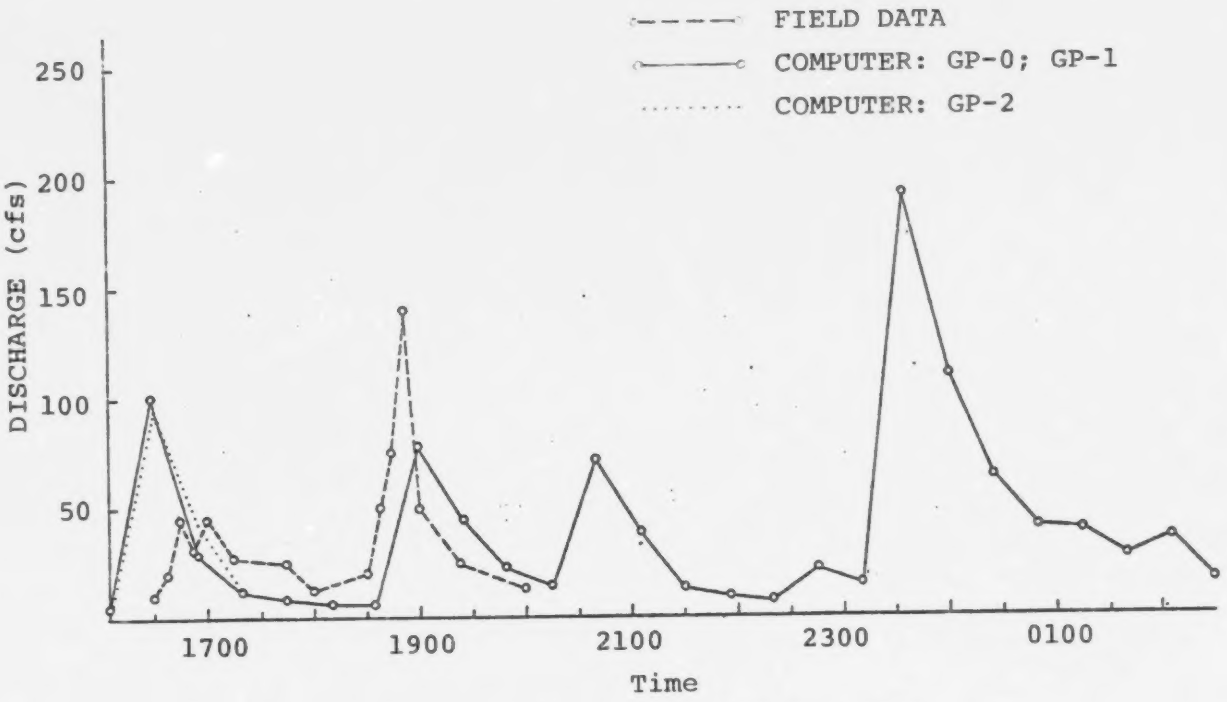
Graph A-33. SUSPENDED SOLIDS VS. TIME
OUTLET MONITORING STATION
MAY 13, 1971



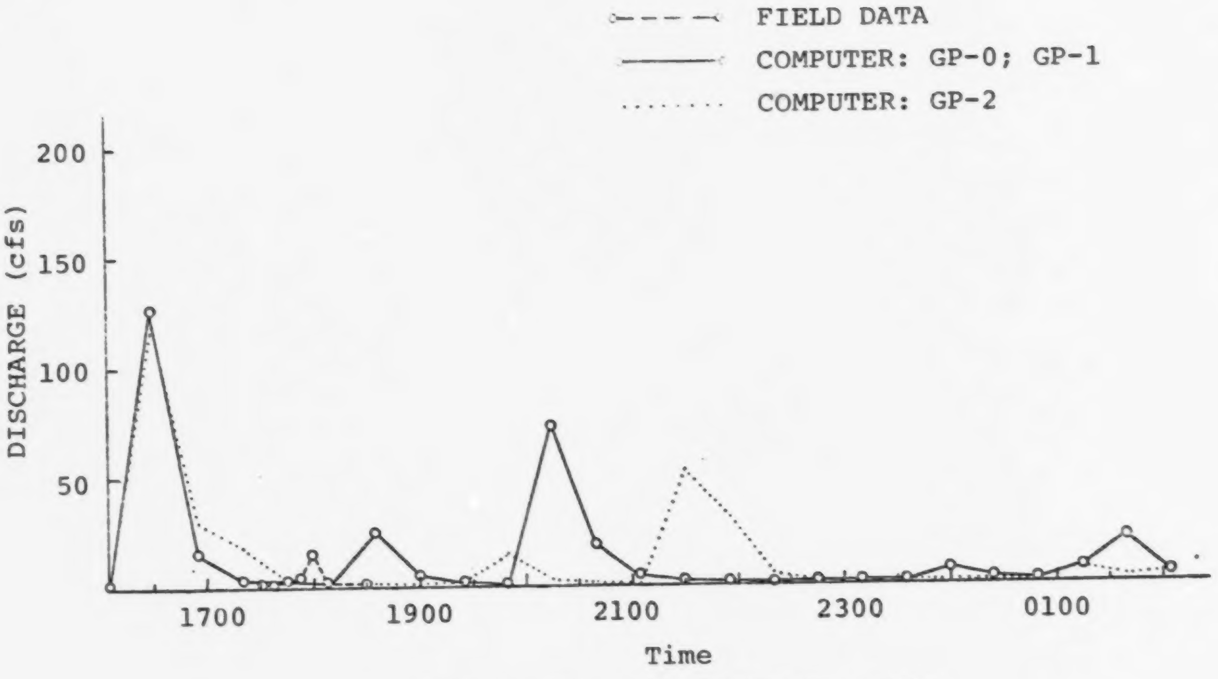
Graph A-34. BOD VS. TIME
OUTLET MONITORING STATION
MAY 13, 1971



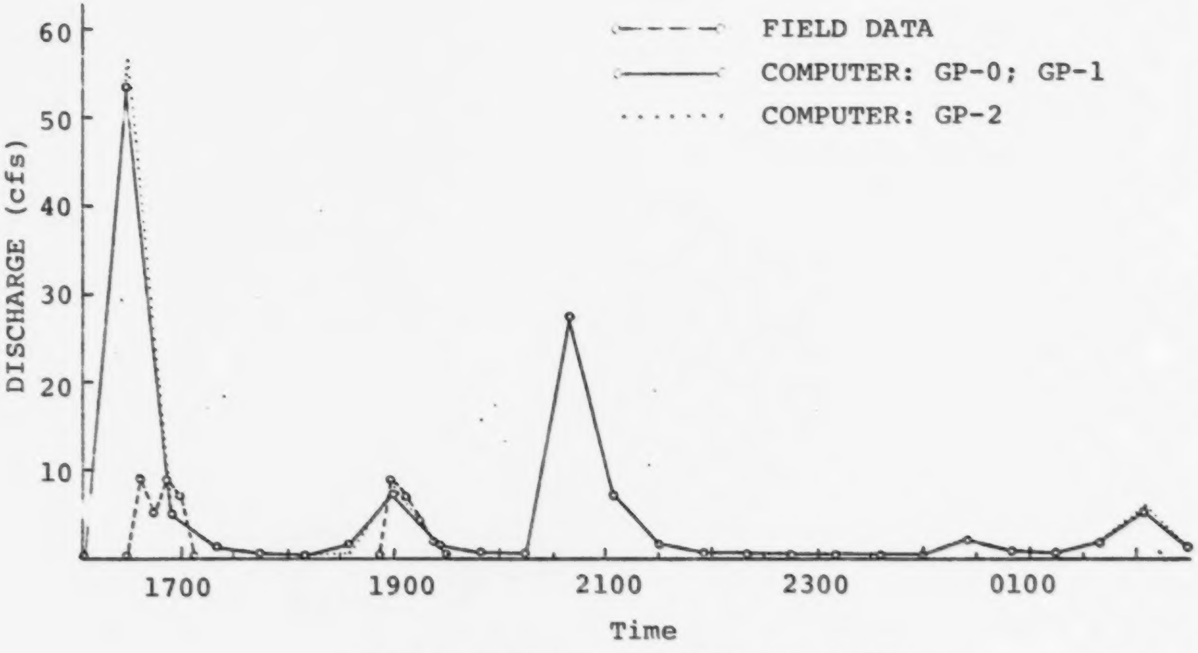
Graph A-35. DISCHARGE VS. TIME
OUTLET MONITORING STATION
AUG. 25-26, 1971



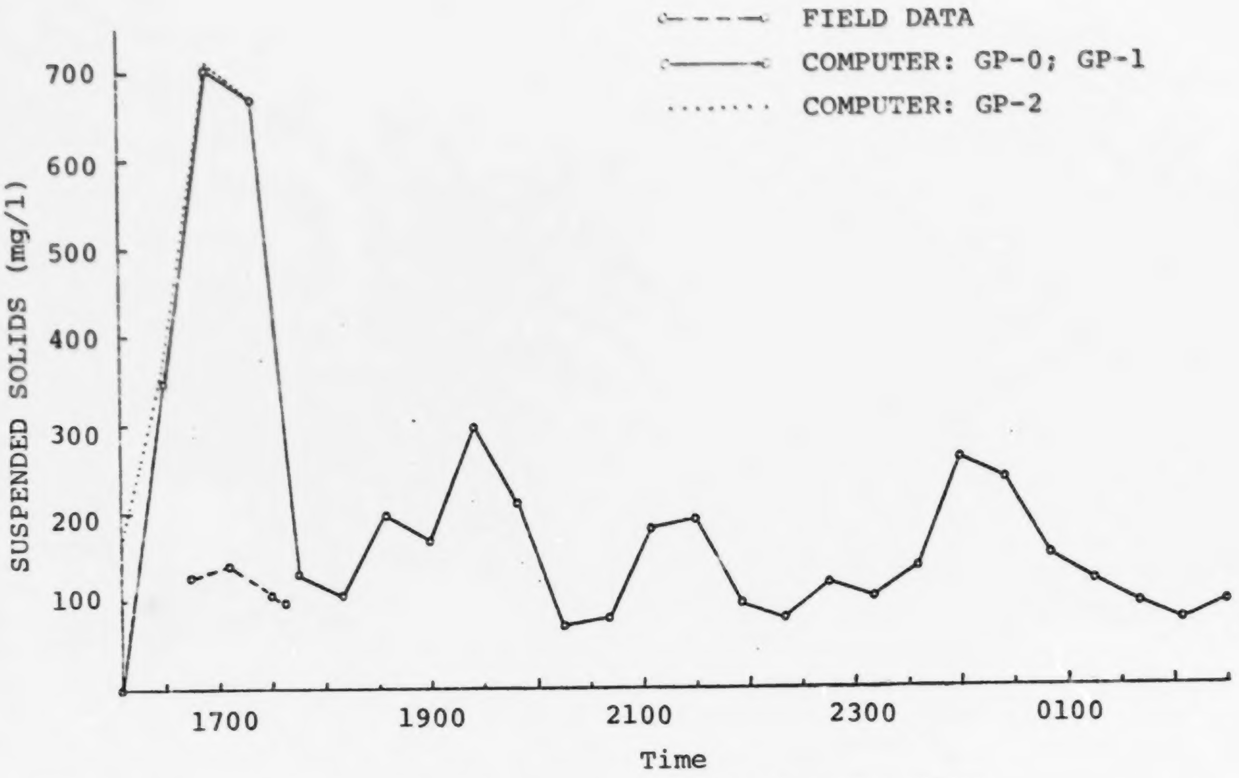
Graph A-36. DISCHARGE VS. TIME
LONGVIEW 11'x9' MONITORING STATION
AUG. 25-26, 1971



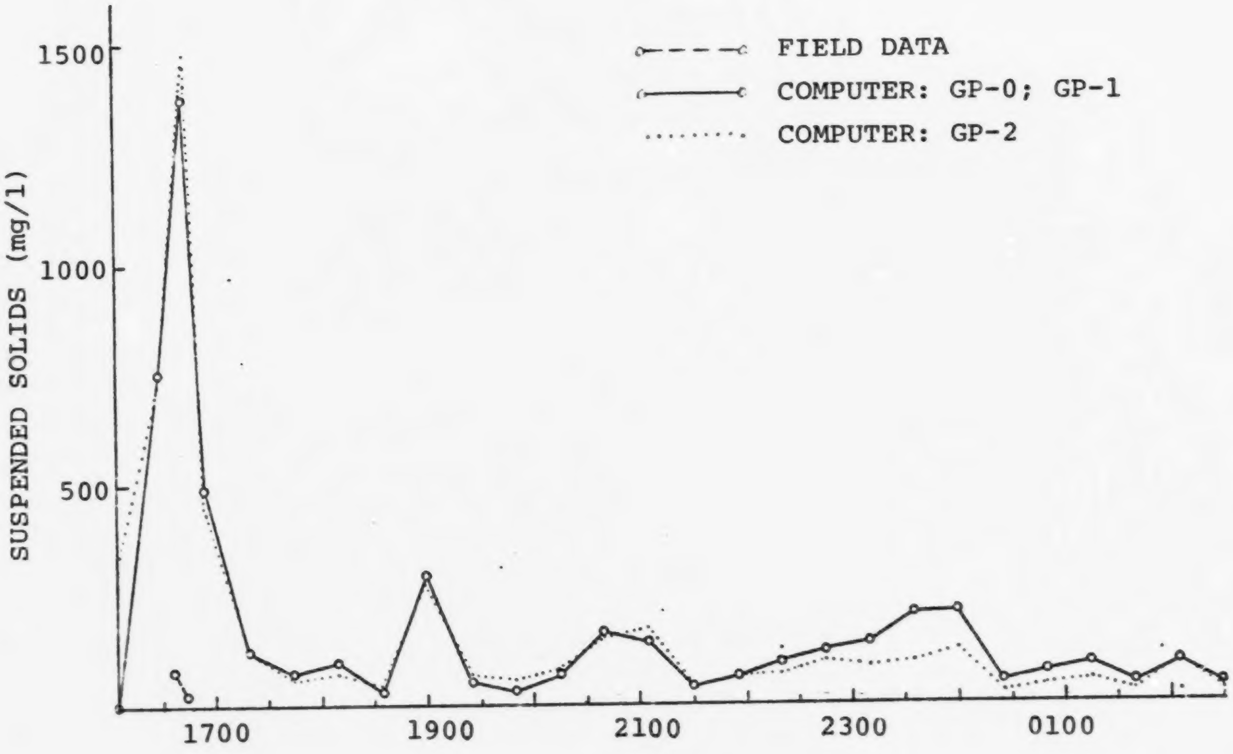
Graph A-37. DISCHARGE VS. TIME
BANK 78" MONITORING STATION
AUG. 25-26, 1971



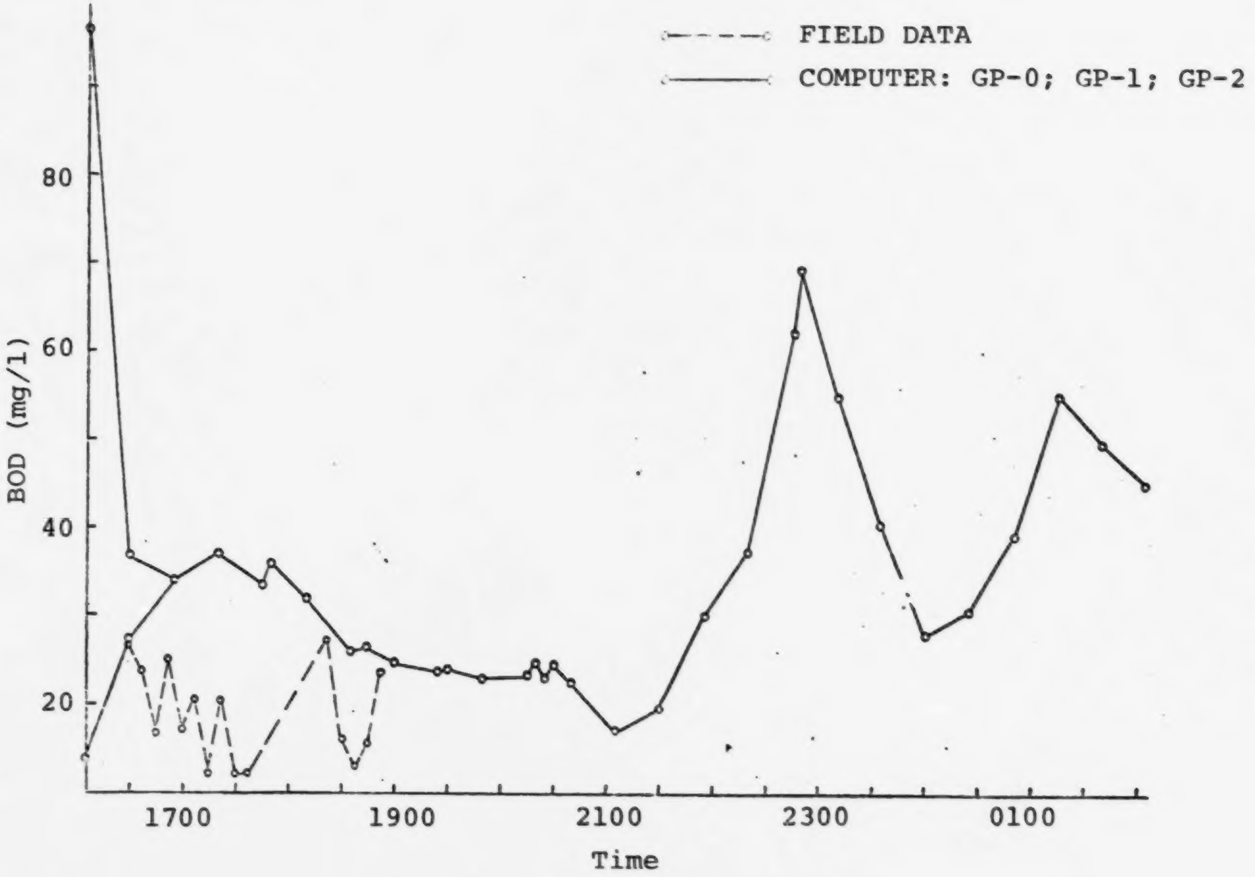
Graph A-38. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
AUG. 25-26, 1971



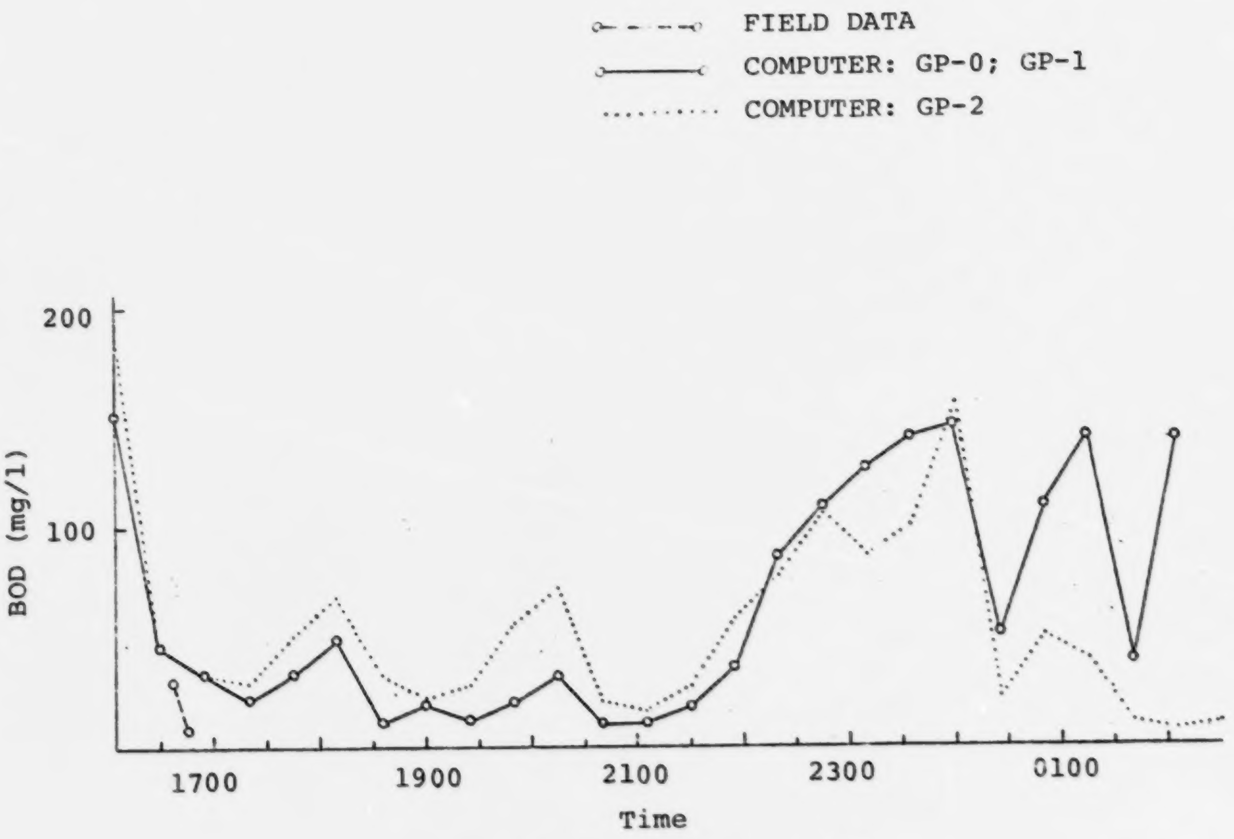
Graph A-39. SUSPENDED SOLIDS VS. TIME
OUTLET MONITORING STATION
AUG. 25-26, 1971



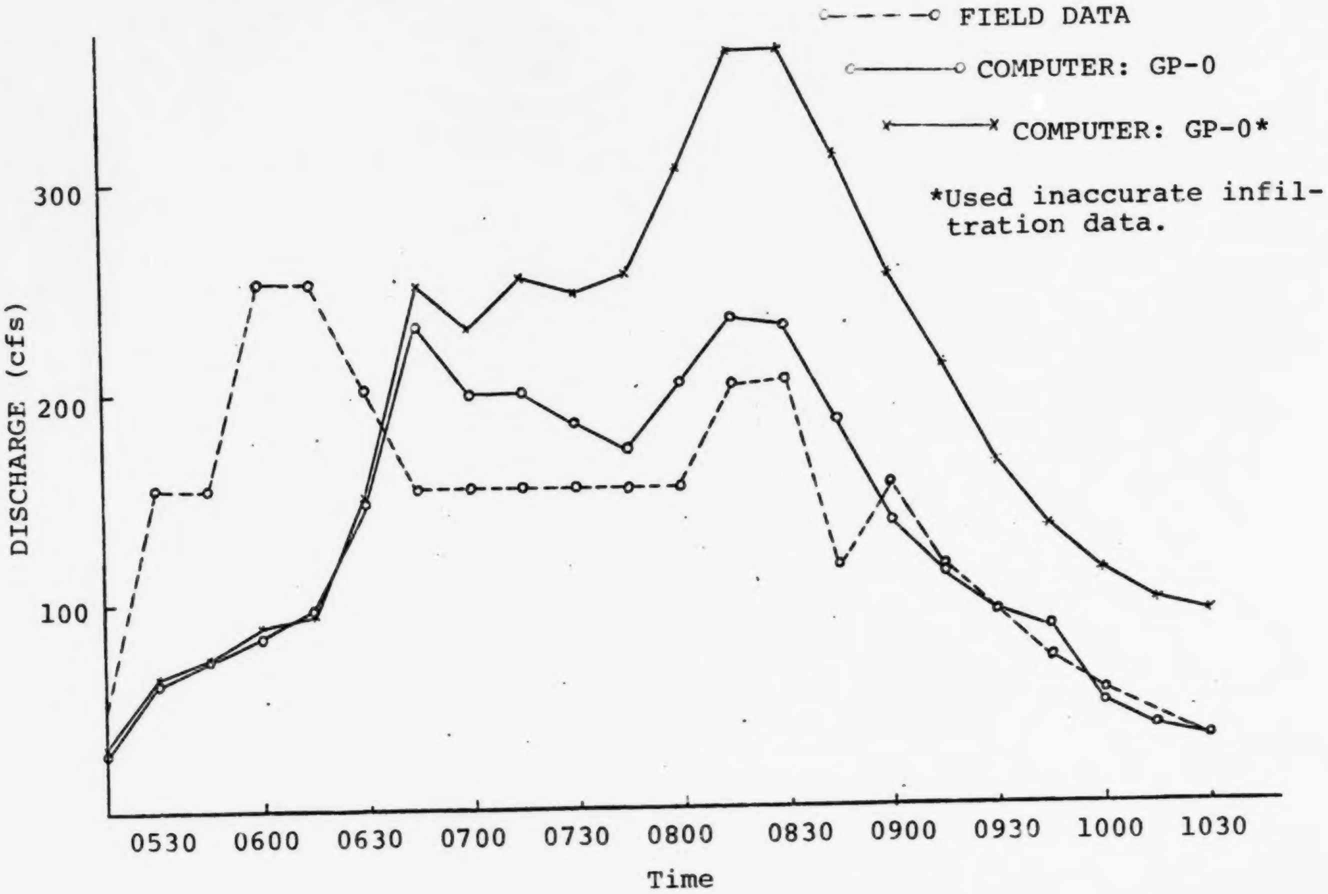
Graph A-40. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
AUG. 25-26, 1971



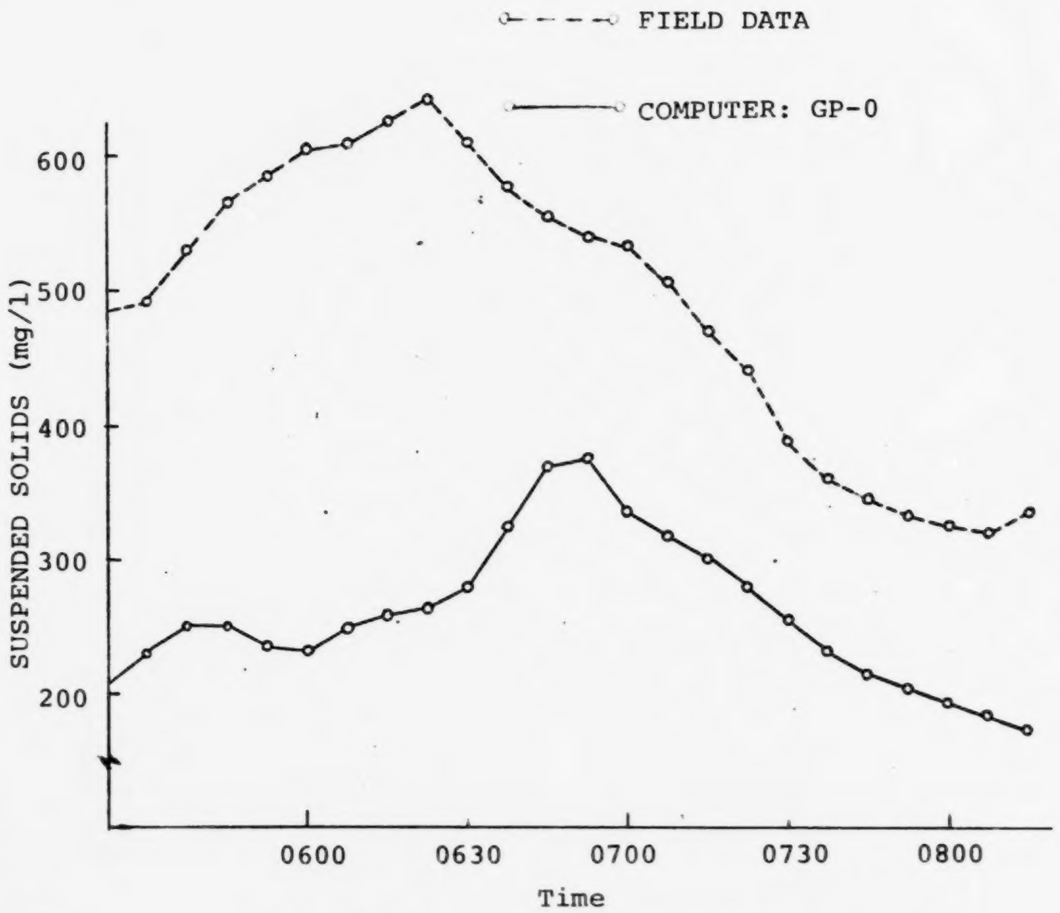
Graph A-41. BOD VS. TIME
OUTLET MONITORING STATION
AUG. 25-26, 1971



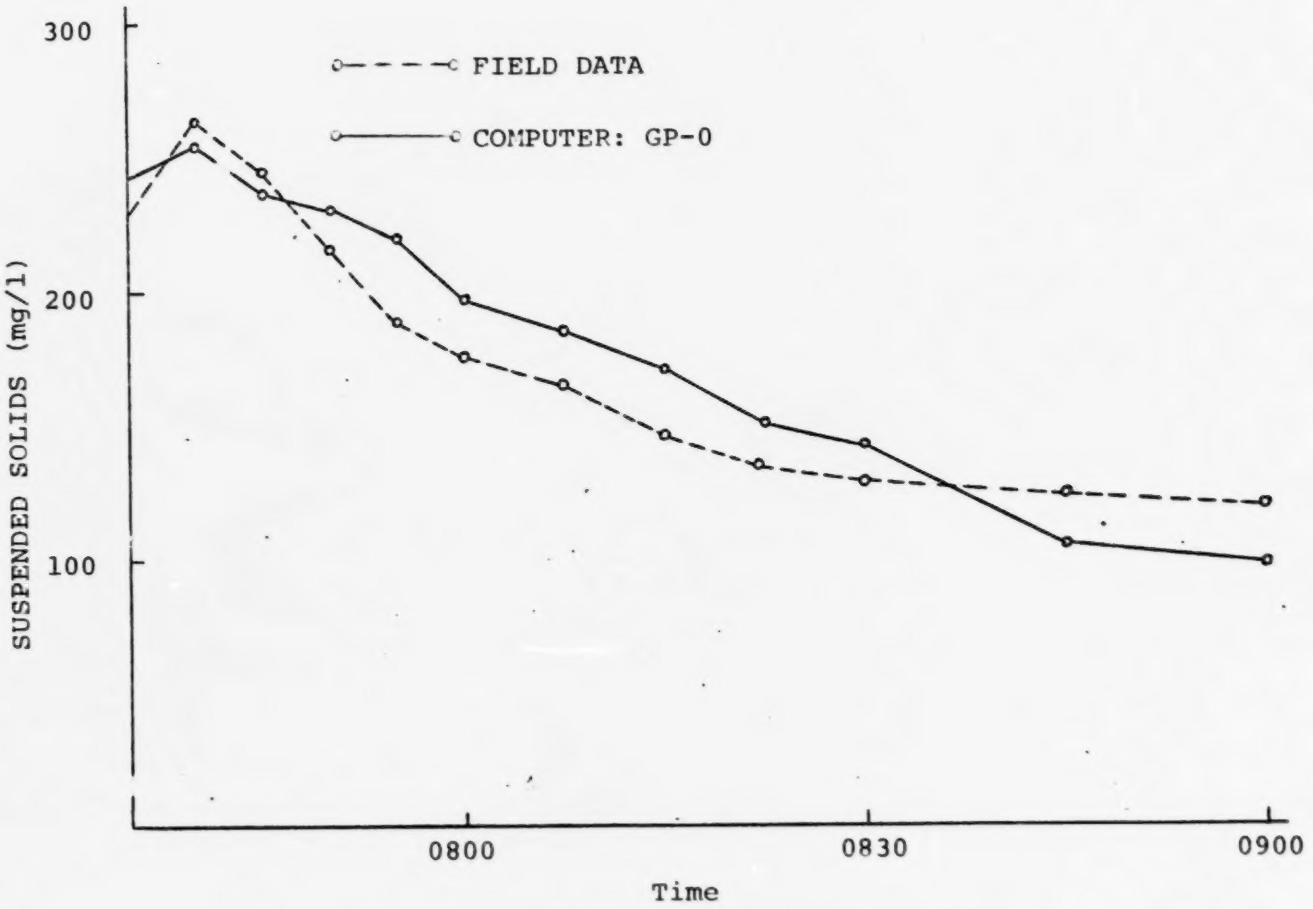
Graph A-42. BOD VS. TIME
BANK 54" MONITORING STATION
AUG. 25-26, 1971



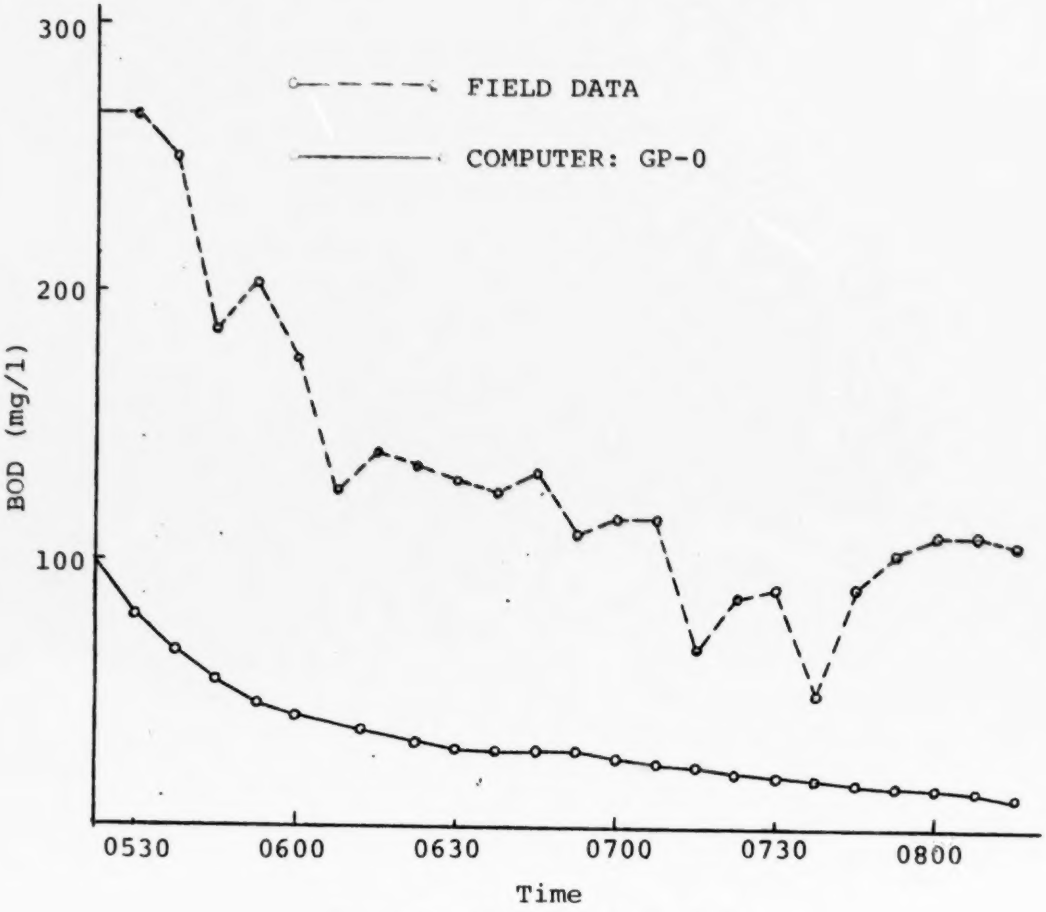
Graph A-43. DISCHARGE VS. TIME
OUTLET MONITORING STATION
OCT. 20, 1970



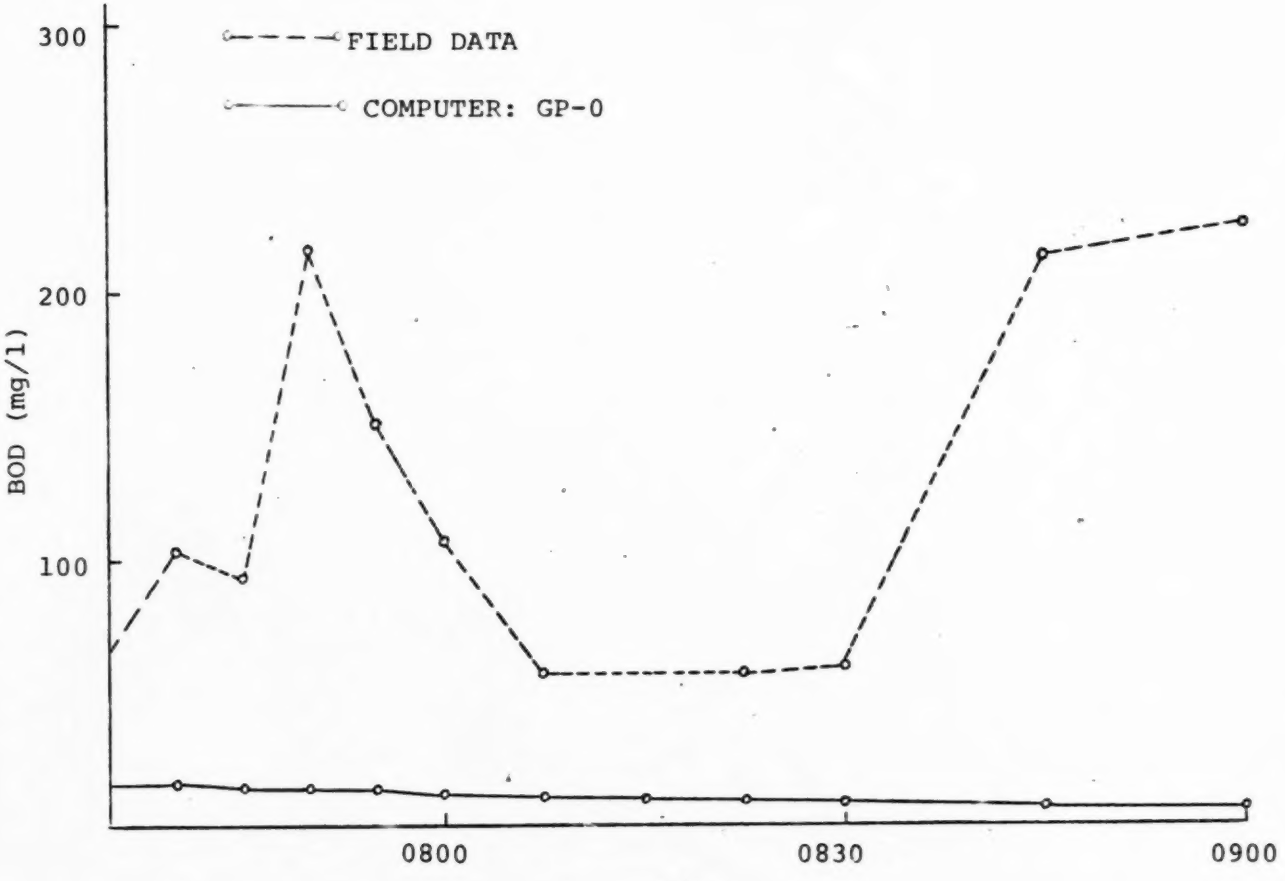
Graph A-44. SUSPENDED SOLIDS VS. TIME
OUTLET MONITORING STATION
OCT. 20, 1970



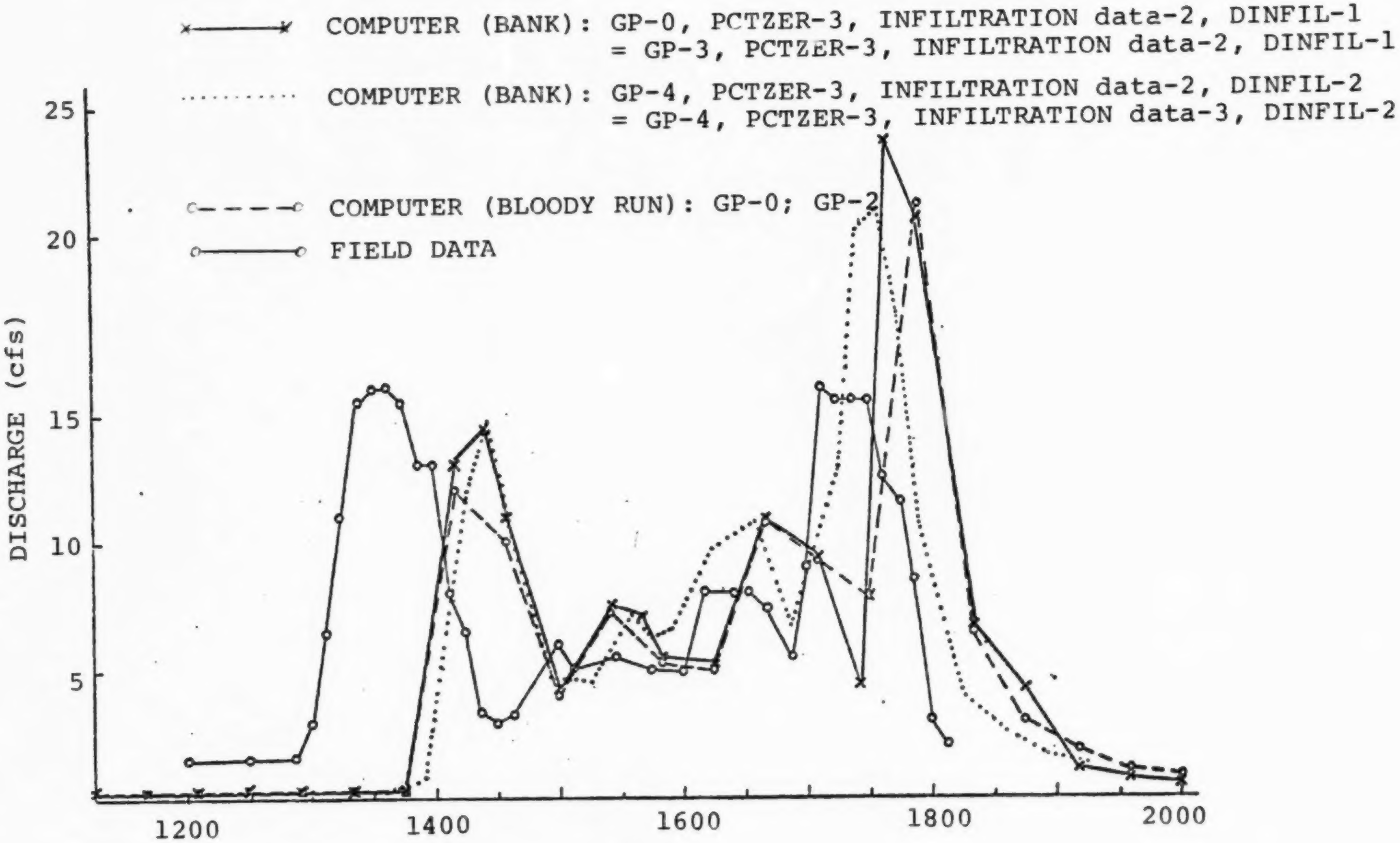
Graph A-45. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
OCT. 20, 1970



Graph A-46. BOD VS. TIME
OUTLET MONITORING STATION
OCT. 20, 1970

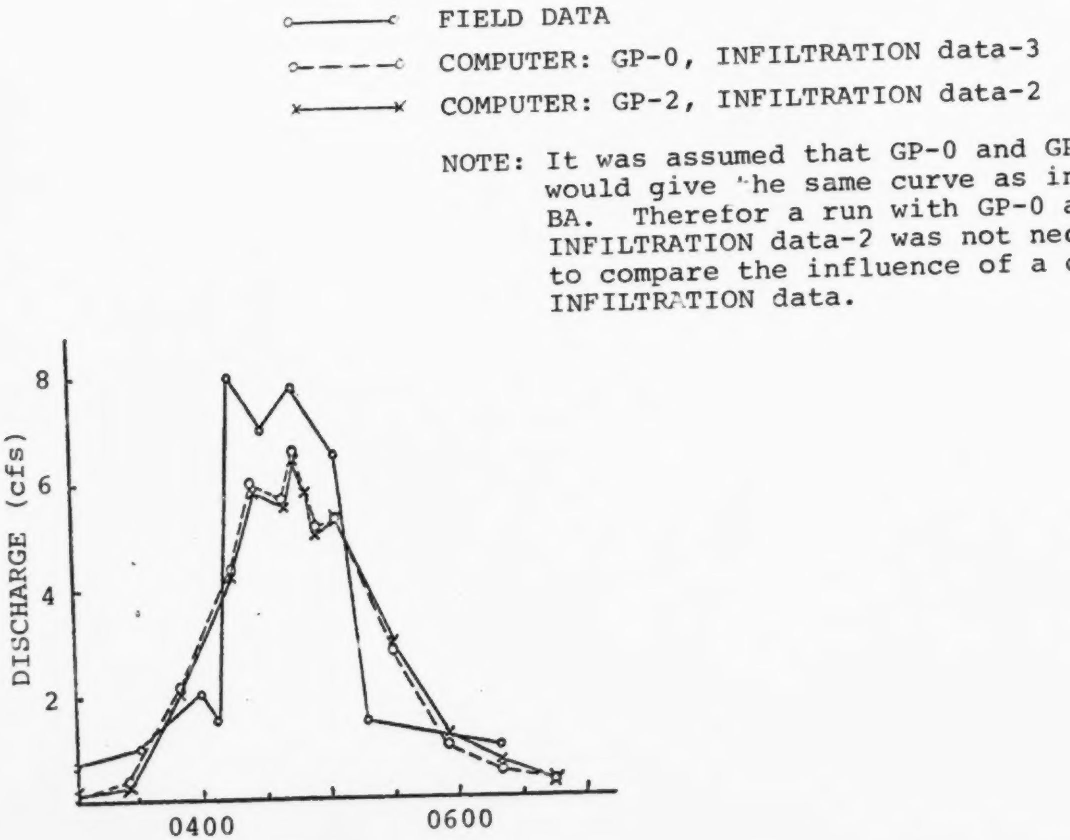


Graph A-47. BOD VS. TIME
BANK 54" MONITORING STATION
OCT. 20, 1970

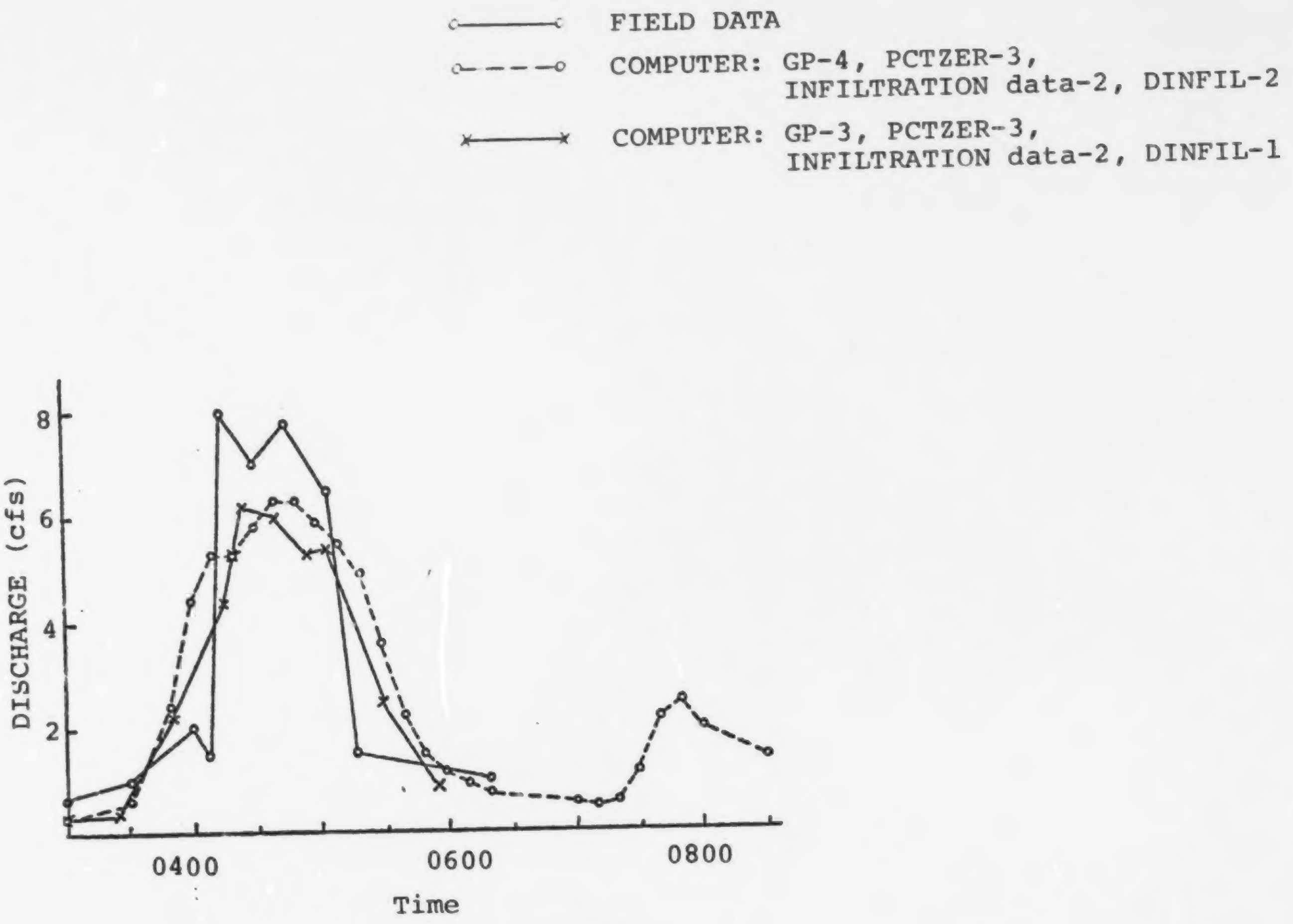


Graph BA. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
MAY 7, 1971

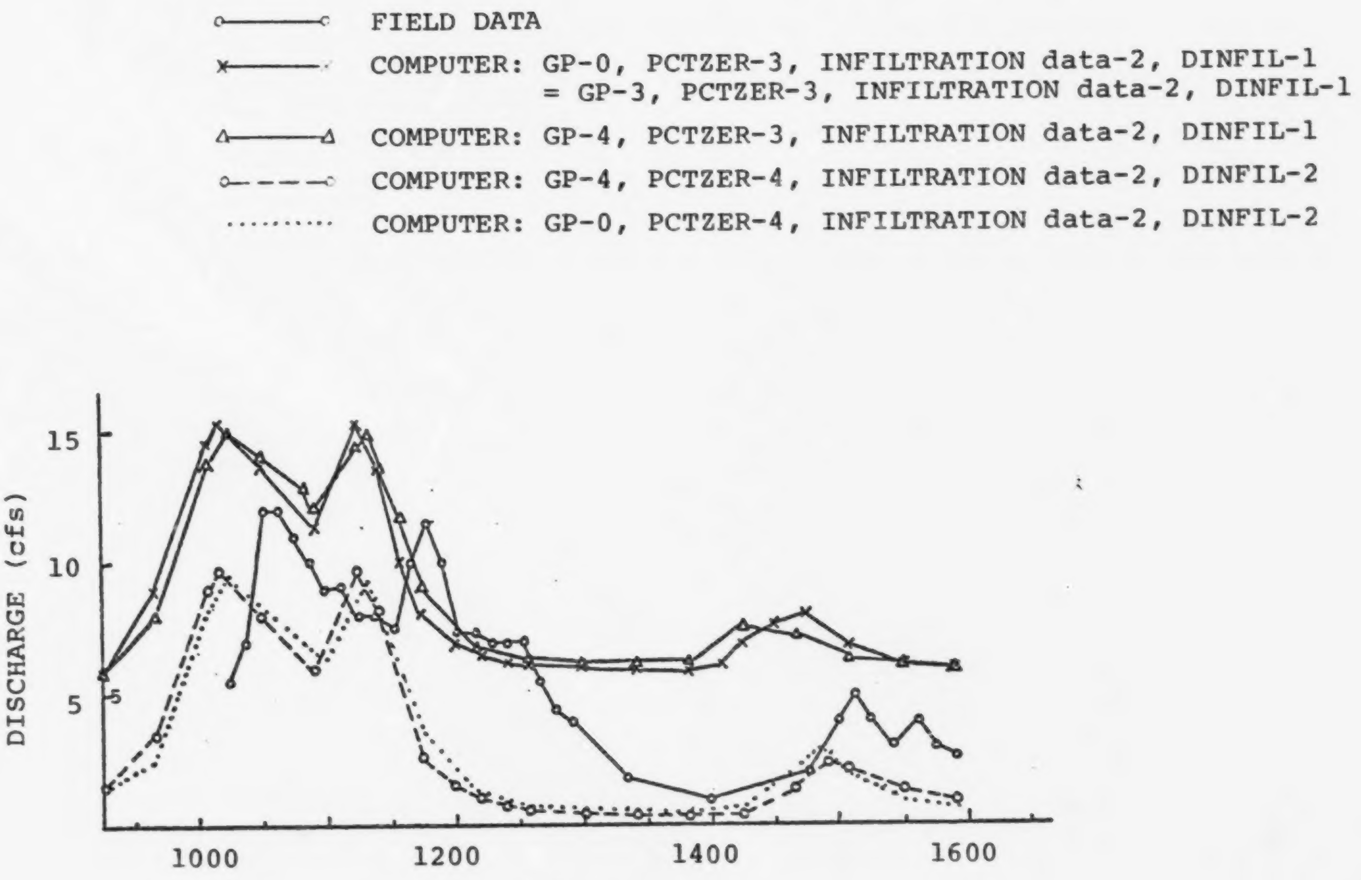
NOTE: It was assumed that GP-0 and GP-2 would give the same curve as in Graph BA. Therefor a run with GP-0 and INFILTRATION data-2 was not necessary to compare the influence of a change in INFILTRATION data.



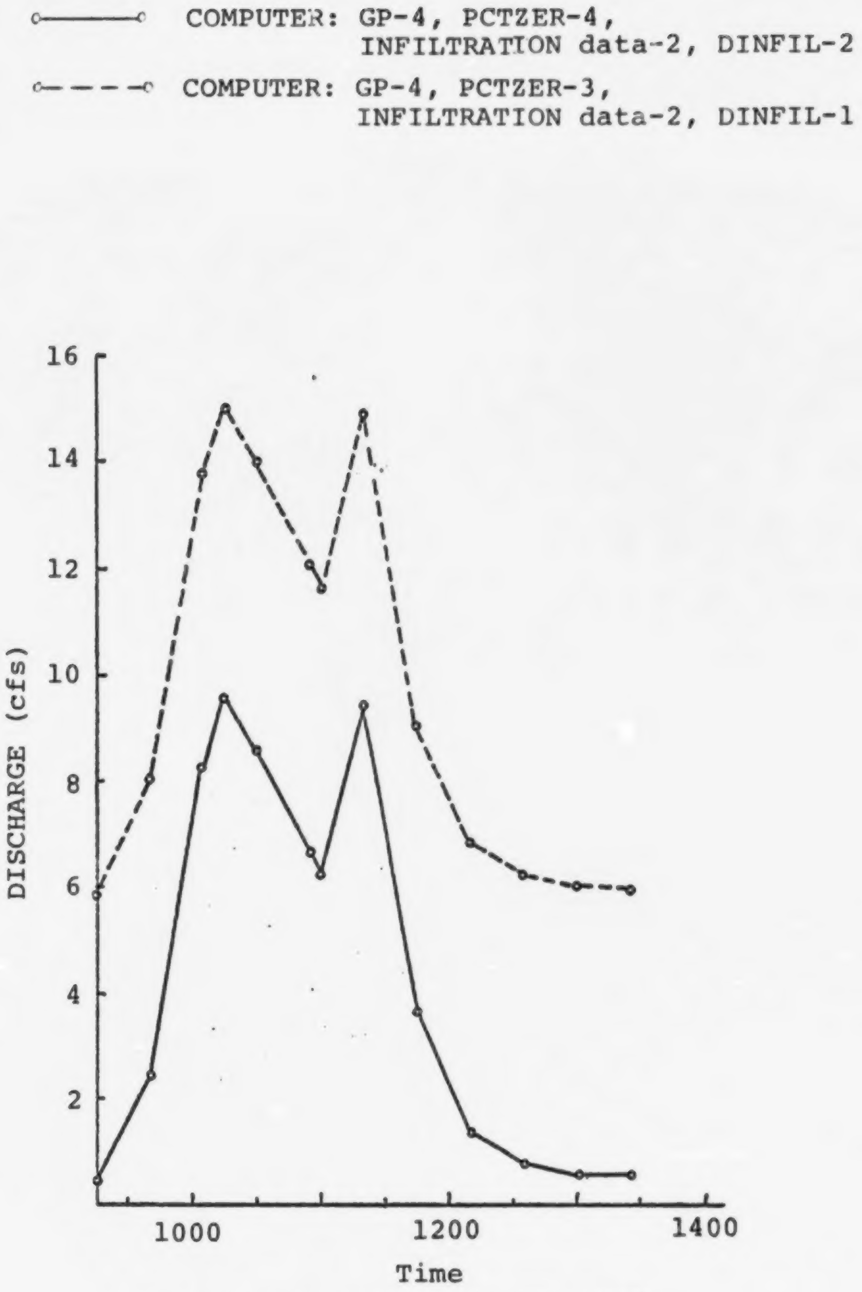
Graph BB. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
MAY 13, 1971



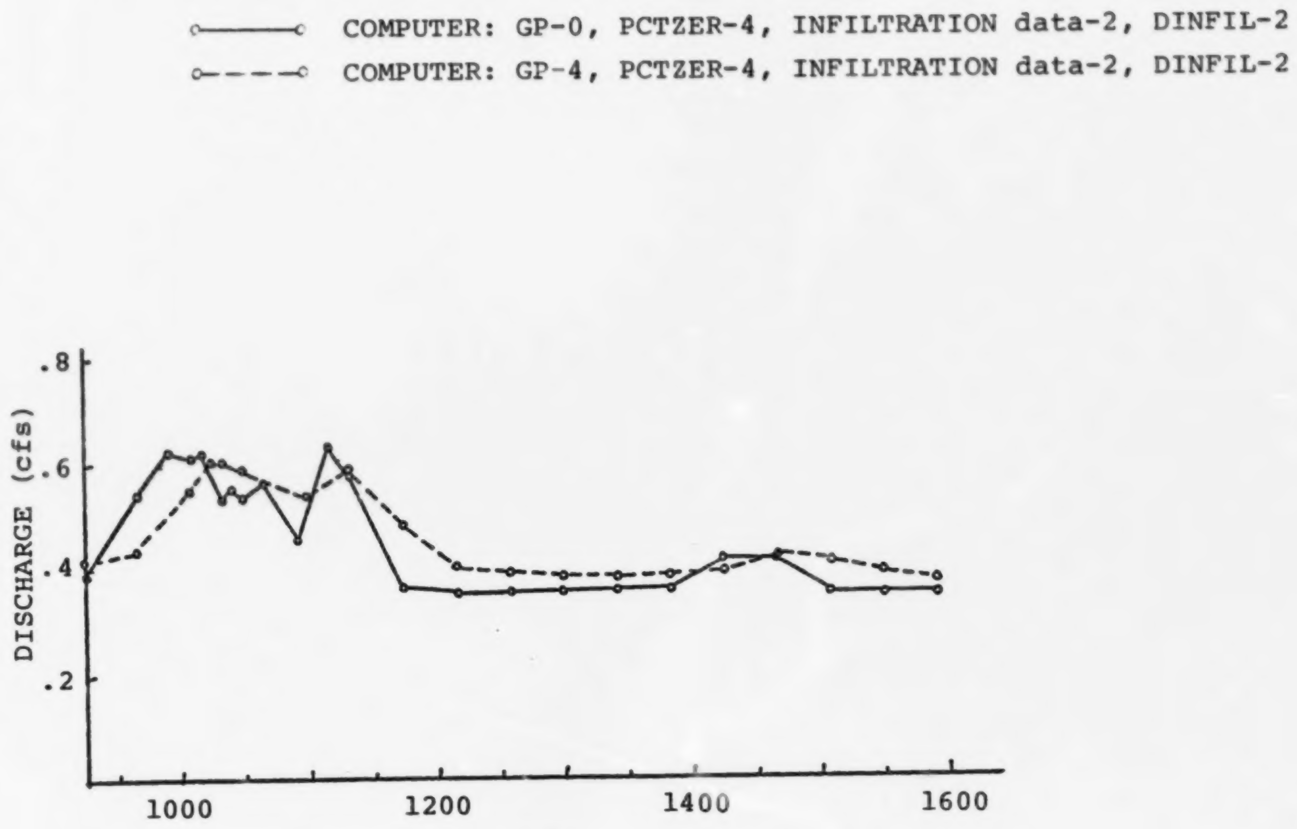
Graph BC. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
MAY 13, 1971



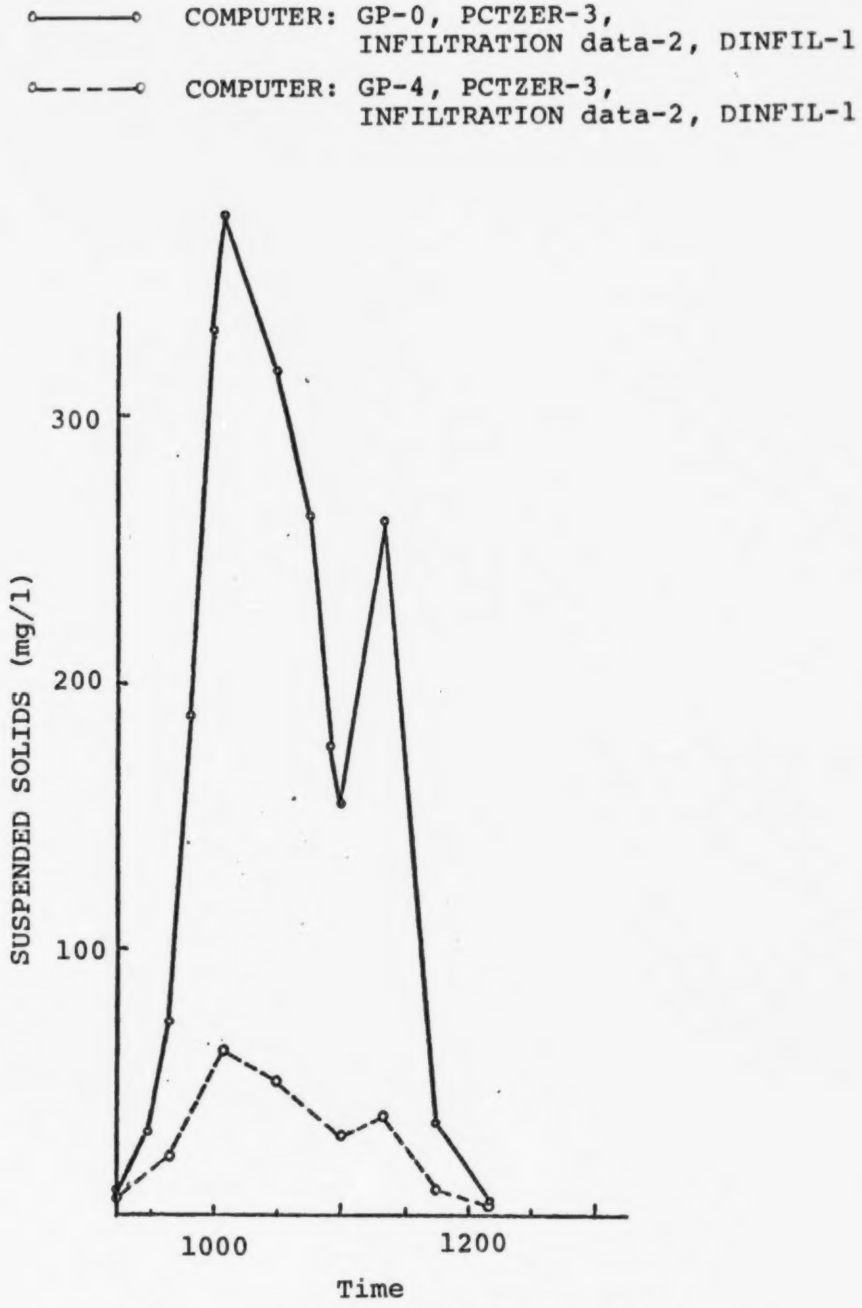
Graph BD. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



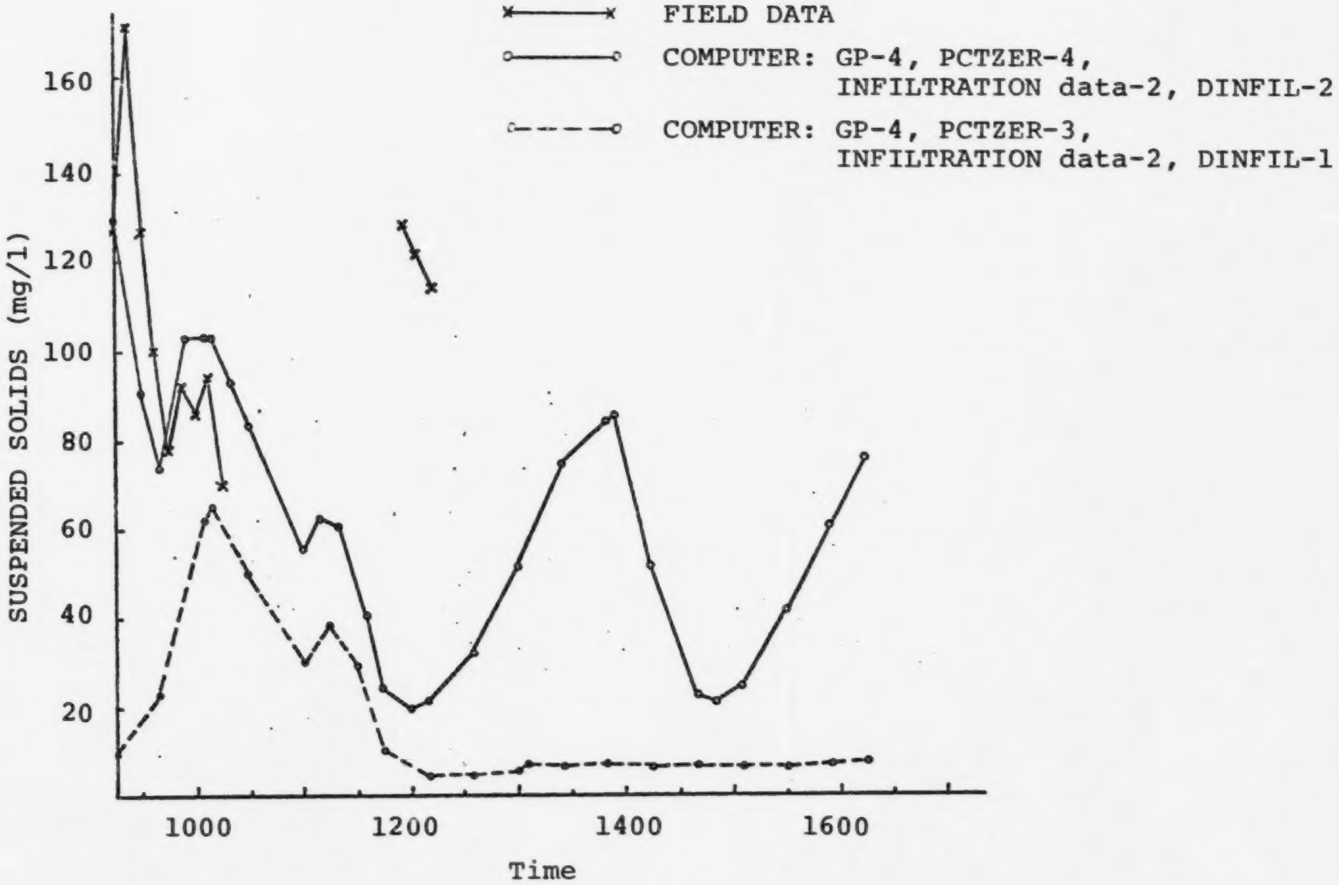
Graph BE. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



Graph BF. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971

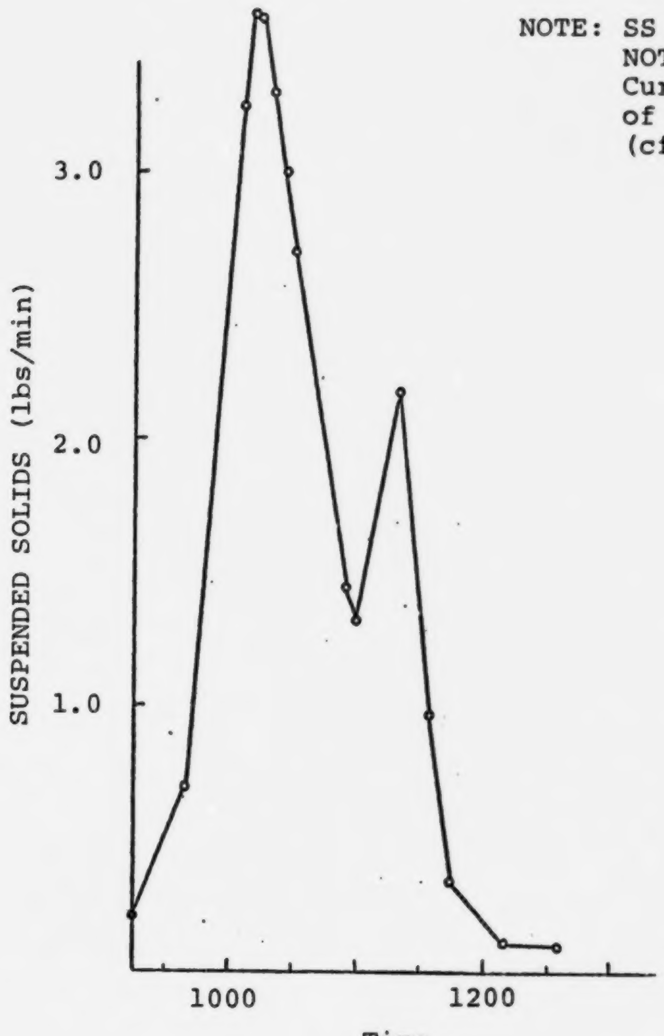


Graph BG. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



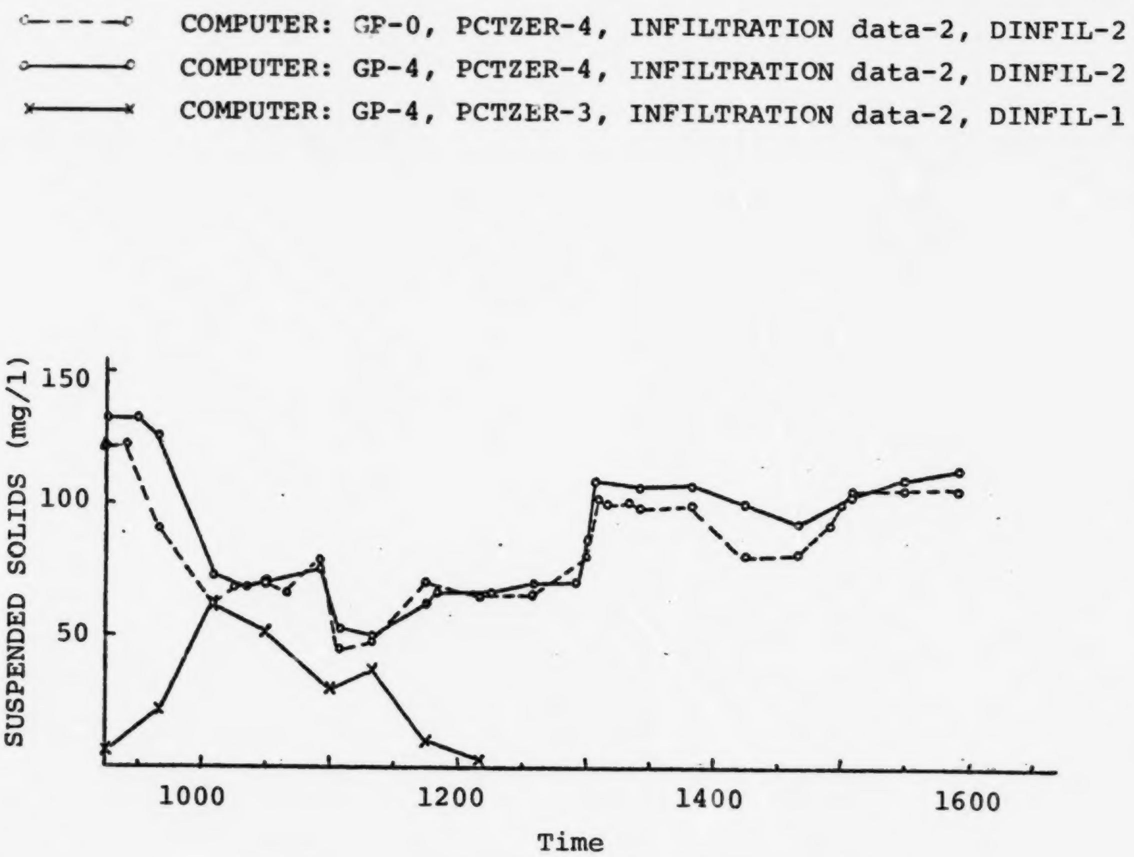
Graph BH. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971

○——○ COMPUTER: GP-4, PCTZER-3, INFILTRATION data-2, DINFIL-1
= GP-4, PCTZER-4, INFILTRATION data-2, DINFIL-2

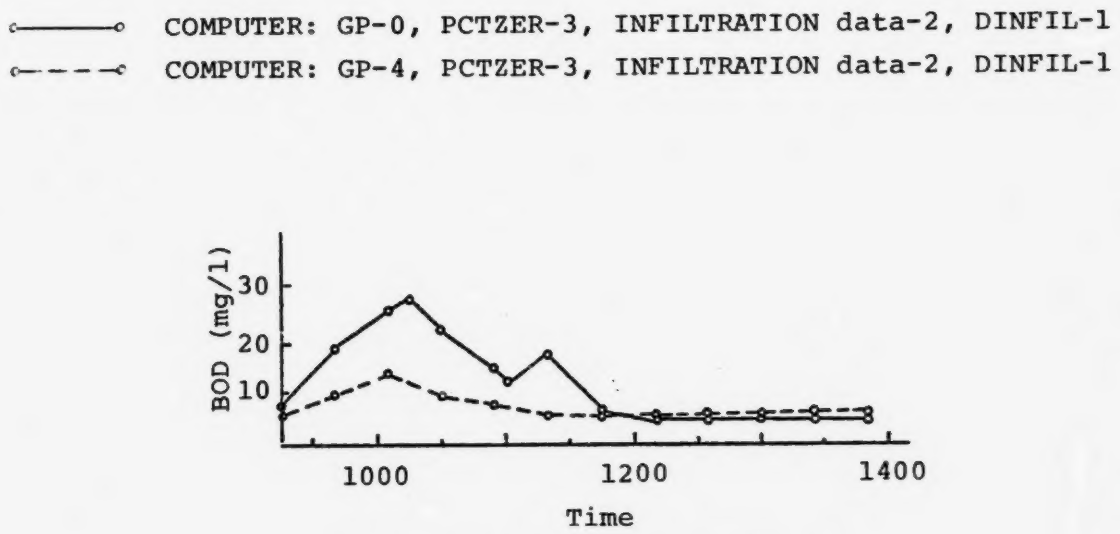


NOTE: SS in lbs/min,
NOT mg/l.
Curves for both sets
of input were identical
(cf. Graph BH).

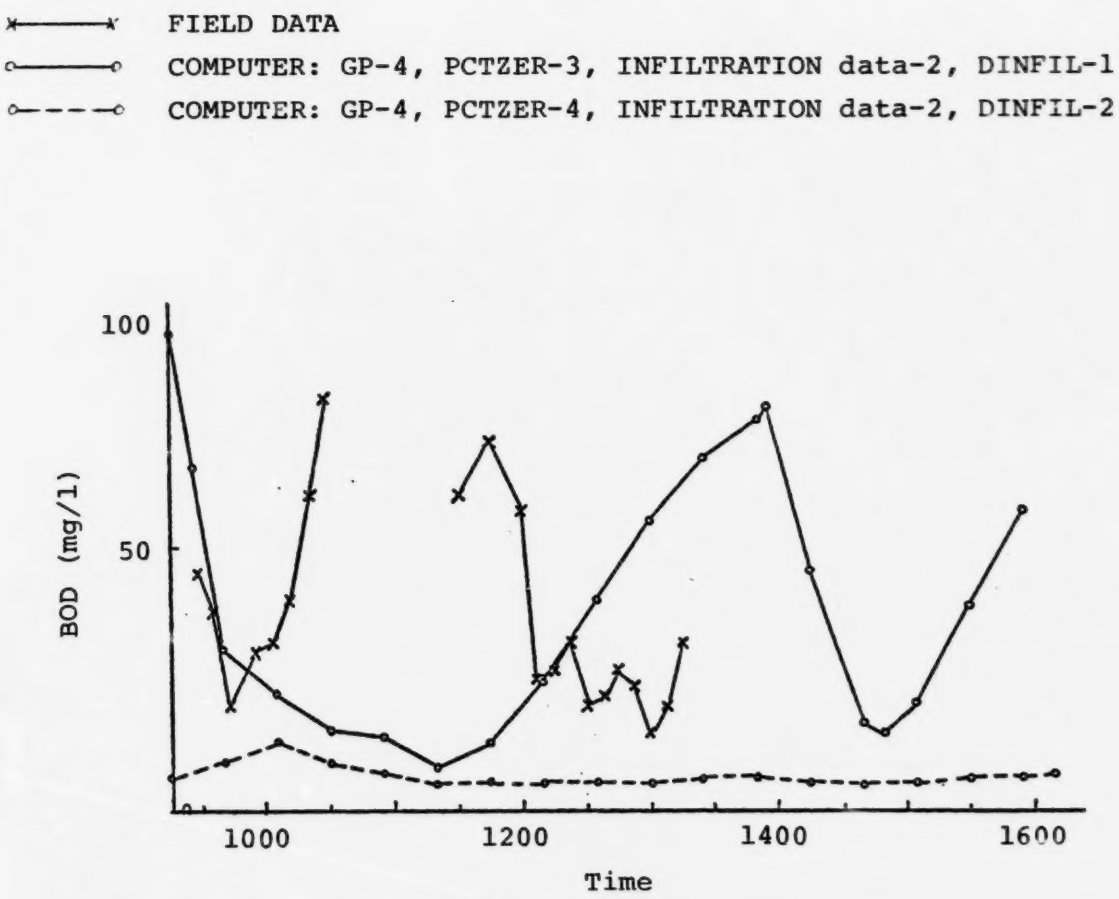
Graph BI. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



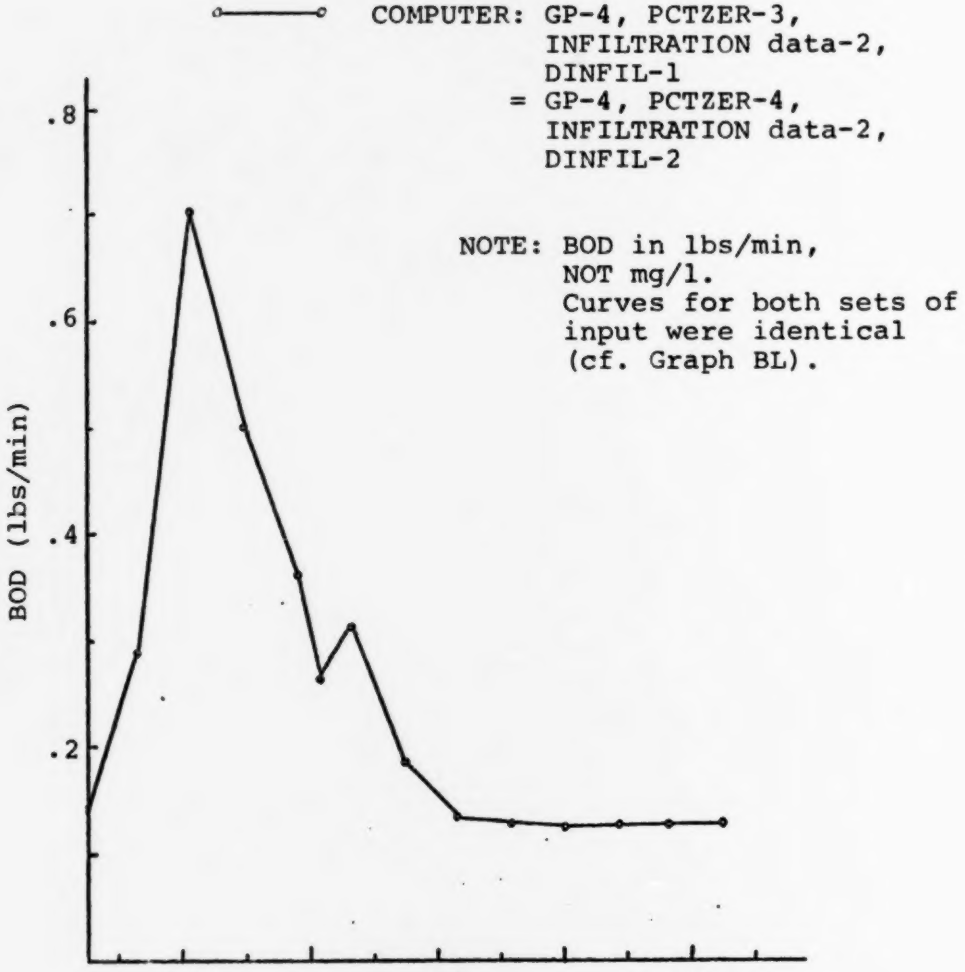
Graph BJ. SUSPENDED SOLIDS VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



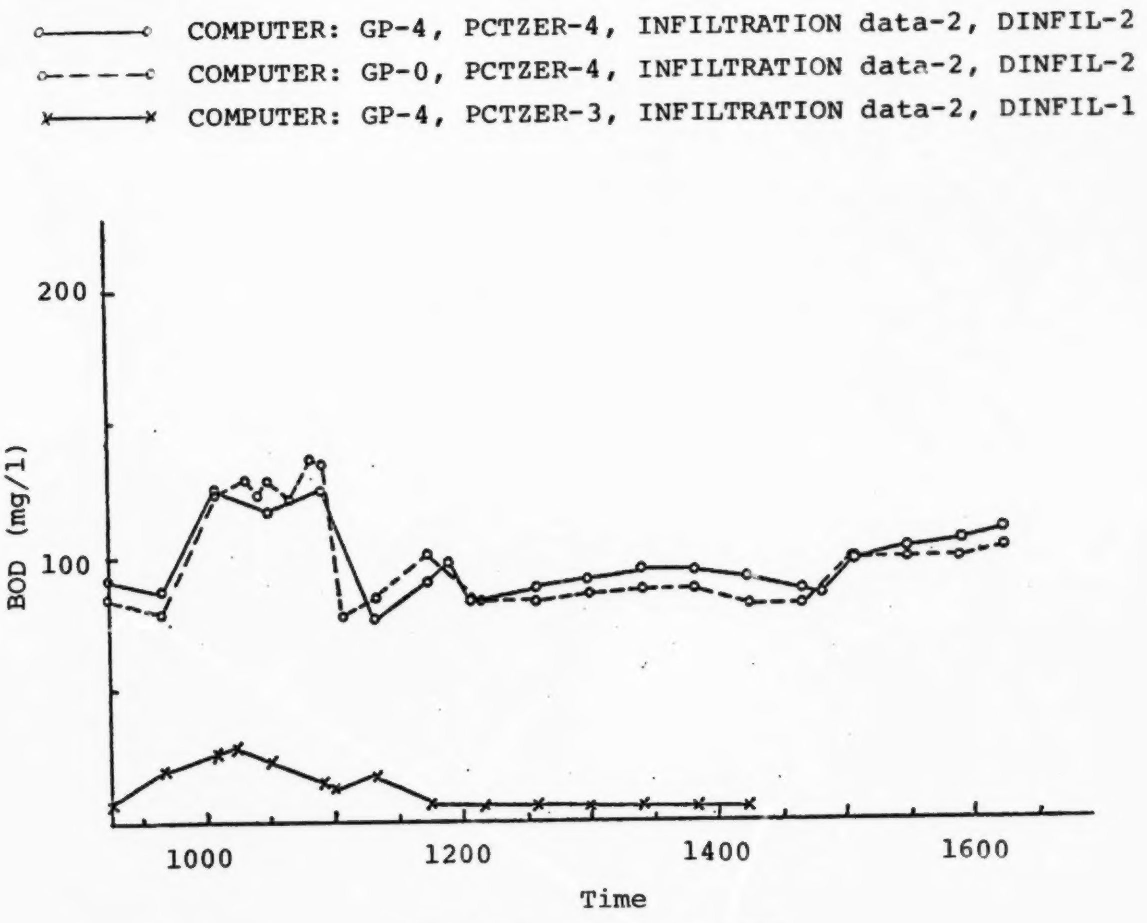
Graph BK. BOD VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



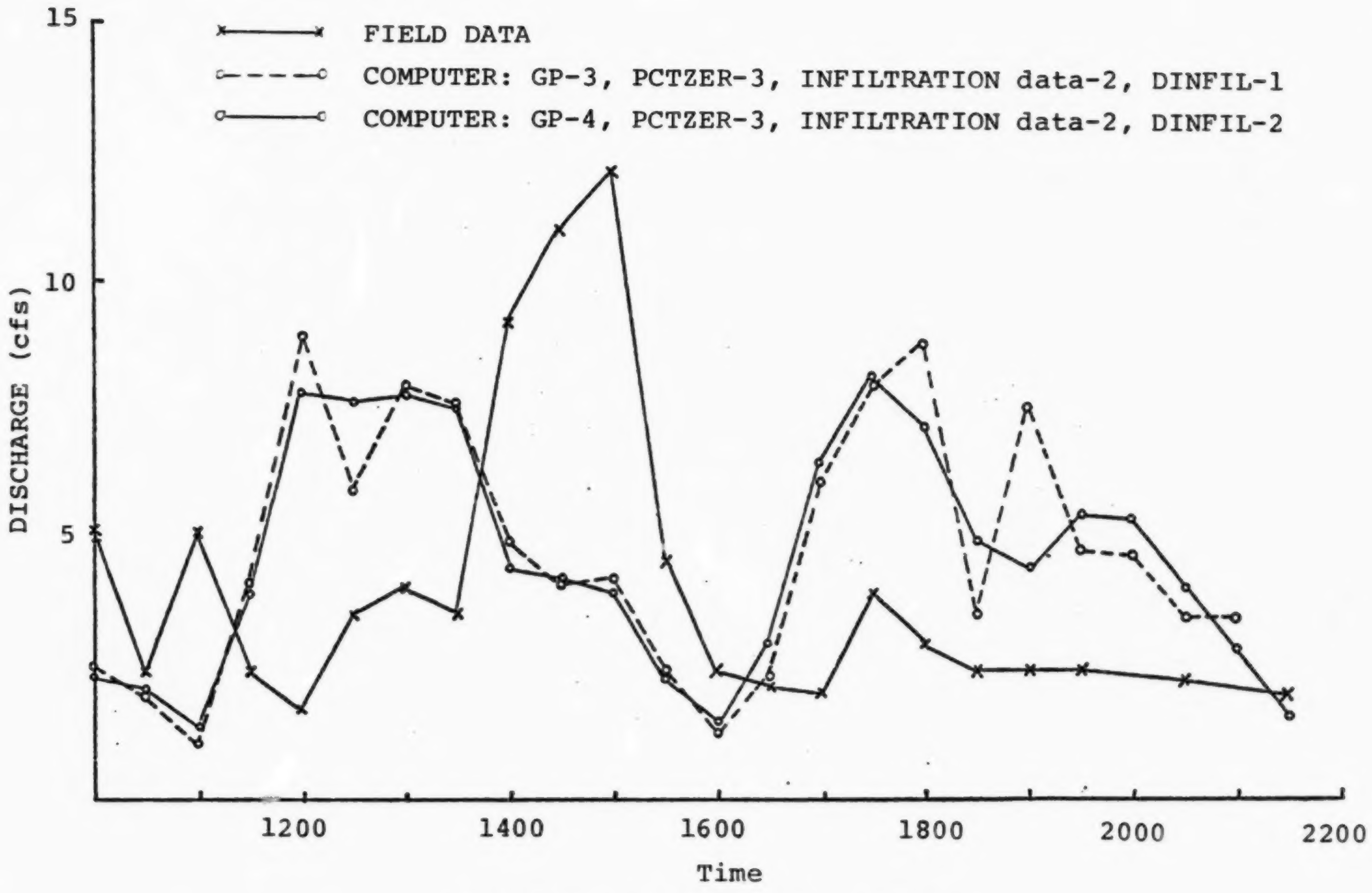
Graph BL. BOD VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



Graph BM. BOD VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



Graph BN. BOD VS. TIME
BANK 54" MONITORING STATION
AUG. 4, 1971



Graph BO. DISCHARGE VS. TIME
BANK 54" MONITORING STATION
DEC. 6, 1971

APPENDIX D

BLOODY RUN WATERSHED DATA
(WITHOUT GUTTER-PIPE CARDS)

STORM OF NOVEMBER 14, 1970

```
 INCINNATI BLOODY RUN SEWER SYSTEM
   1     2375.80      13.50     10      2570.0      1     2570.14
 OVEMBER 14, 1970                 0.51
    0    8    8    9   10   10   11    0   12   12   14    0   15   15   16
    1    2    3    4   13
 ATERSHED
 BLOODY RUN WATERSHED, CINCINNATI       COMBINED SEWER SYSTEM
     DATA FILE 3H5MC4
    38  146 073.   5.0      3  42.
  120  5.0
.0001.0001.0001 0.08 0.08 0.070.0360.0480.036 0.12
 0.06 0.06 0.06 0.06 0.18 0.06 0.06 0.06 0.080.042
 .0360.0420.0420.0360.0420.0420.0360.0420.0420.036
 0.04 0.04 0.04 0.04 0.04 0.04 0.12 0.12 0.120.036
 .0360.048 0.07 0.08 0.08 0.07 0.08 0.080.0480.048
 .0480.0480.048 0.060.0240.0240.0240.0240.0240.024
 .0240.0240.0240.0240.024 .036 0.02 0.02 0.02 0.02
 0.02 0.02 0.02 0.02 0.02 0.02 0.02 0.02 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
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 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
 0.12 0.12 0.12 0.00 0.12 0.06 0.06 0.06 0.06 0.12
 0.12 0.12 0.12 0.06 0.06 0.24 0.06 0.06 0.06 0.06
 0.12 0.12 0.12 0.12 0.24 0.12 0.24 0.06 0.12 0.18
 0.12 0.12 0.12 0.12 0.12 0.18 0.06 0.12 0.12 0.12
 0.12 0.12 0.12 0.12 0.12 0.60 0.12 0.12 0.00 0.06
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 0.12 0.06 0.06 0.12 0.12 0.06 0.06 0.06 0.18 0.06
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 0.06 0.06 0.06 0.06 0.04 0.02 0.04 0.02 0.06 0.02
 0.04 0.06 0.06 0.06 0.06 0.06 0.05 0.02 0.01 0.01
 0.01 0.01 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
.0001.0001.0001.0001.0001.0001.0001.0001 0.12 0.12
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 0.12 0.00 0.00 0.00 0.00 0.00 0.12 0.12 0.12 0.12
 0.00 0.06 0.06 0.06 0.06 0.12 0.12 0.12 0.06 0.06
 0.12 0.12 0.12 0.12 0.00 0.12 0.06 0.06 0.06 0.06
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 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.12 0.06
 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.00 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.12 0.12 0.12 0.12
 0.12 0.06 0.00 0.00 0.00 0.00 0.00 0.00 0.06 0.06
 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
     3    1     11530.176.0 33.7.0310                              3.00 0.52     .00113
     3    2      91138. 68.6 28.7.0384                              3.00 0.52     .00113
     3    3    151262. 73.2 53.5.0442                              3.00 0.52     .00113
     3    4    174112. 59.0 34.7.0301                              3.00 0.52     .00113
     3    5    191558. 38.0 36.8.1120                              3.00 0.52     .00113
     3    6    21 970. 33.4 38.9.1050                              3.00 0.52     .00113
     3    7    25 481. 69.0 73.3.0315                              3.00 0.52     .00113
     3    8    [illegible]                                         3.00 0.52     .00113
```

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```
 3   9   312389.250.2 51.9.0401                           3.00 0.52     .00113
 2  10   332625. 45.2 58.5.0200                           3.00 0.52     .00113
 2  11   351467. 30.4 81.4.0430                           3.00 0.52     .00113
 2  12   372591. 81.6 54.5.0320                           3.00 0.52     .00113
 2  13   391523. 45.8 10.8.0500                           3.00 0.52     .00113
 2  14   41 380. 20.2 34.7.0350                           3.00 0.52     .00113
 2  15  1121167. 72.0 20.7.0370                           3.00 0.52     .00113
 2  16   435515.116.0 45.6.0375                           3.00 0.52     .00113

 2  17   451598. 91.2 64.5.0238                           3.00 0.52     .00113
 2  18  108 830. 14.3 24.5.0400                           3.00 0.52     .00113
 2  19   512653. 50.9 55.8.0220                           3.00 0.52     .00113
 2  20   531646. 42.5 39.7.0150                           3.00 0.52     .00113
 2  21   571798. 36.1 42.3.0420                           3.00 0.52     .00113
 2  22   591545. 39.9 46.4.0250                           3.00 0.52     .00113
 2  23   613571. 87.1 60.8.0570                           3.00 0.52     .00113
 2  24   652461. 86.3 38.7.0530                           3.00 0.52     .00113
 2  25   71 635. 30.1 44.6.0850                           3.00 0.52     .00113
 2  26   751310. 29.9 71.7.0640                           3.00 0.52     .00113
 2  27   771072. 21.9 56.4.0590                           3.00 0.52     .00113
 3  28   872335. 60.3 48.7.0430                           3.00 0.52     .00113
 2  29   892091. 46.0 39.8.0360                           3.00 0.52     .00113
 3  30   911954. 50.6 29.7.0660                           3.00 0.52     .00113
 2  31   931243. 60.0  5.0.0495                           3.00 0.52     .00113
 2  32   95 904. 54.0  4.9.0250                           3.00 0.52     .00113
 1  33  114 725. 25.0 16.8.0270                           3.00 0.52     .00113
 1  34   97 896. 50.1 10.9.0235                           3.00 0.52     .00113
 1  35  116 638. 62.5 39.5.0125                           3.00 0.52     .00113
 1  36  1017600.122.0 49.7.0659                           3.00 0.52     .00113
 1  37  1051156. 49.0 33.8.0710                           3.00 0.52     .00113
 2  38  109 210.  3.6 95.0.0400                           3.00 0.52     .00113

 38
  1    9   15   17   19   21   25   27   31   33   35   37   39   41   43  112
 45   51   53  106   57   59   61   65   71   75   77   87   89   91   93   95
114   97  116  101  105  109
  0    0
  0    0    0
 71   38  5.0   07   30  140    0
    10.        21.    1
     1.       125.        100.
  1    1    1      105.6      454.
  2    1    5       52.8        1.
  3    1    3       17.6       47.
  4    9    1       34.3      147.
  5    9    5       34.3        1.
  6   15    1       58.6      252.
  7   15    3       14.6       38.
  8   17    3       14.8       39.
  9   17    1       44.2      190.
 10   19    1       38.0      164.
 11   21    1       20.0       86.
 12   21    5       13.4        1.
 13   25    3       69.0      180.
 14   27    4       43.0       43.
 15   27    2       10.7       46.
 16   31    3       37.5       98.
 17   31    1      212.7      916.
 19   33    1       24.1      117.
 20   33    2       21.1       78.
 21   35    3       23.0       60.
 23   35    1      3.8      16.
 24   37    1     81.8     352.
 25   39    5     37.4       1.
 26   39    3     12.4      32.
 27   41    2     13.4      58.
 28   41    3      7.1      19.
 31   43    1     58.0     250.
 32   43    3     58.0     151.


 29  112    5     43.2       1.
 30  112    3     28.8      75.
 33   45    3     36.2      96.
 34   45    5     27.5       1.
 35   45    2     27.5     108.
 38   51    1     56.9     254.
 39   53    1     42.5     183.
 36  108    5     14.3       1.
 40   57    1     10.8      46.
 41   57    5     14.5       1.
 42   57    3     10.8      28.
 43   59    2     29.9     129.
 44   59    5     10.0       1.
 45   61    1     21.8      94.
 46   61    3     47.9     125.
 47   61    2     17.4      75.
 48   65    2     22.1      95.
 49   65    1     66.2     285.
 50   71    5     10.5       1.
 51   71    4     19.6      20.
 52   75    4     29.9      30.
 53   77    2     10.9      47.
 54   77    1     11.0      47.
 55   87    1     36.1     155.
 56   87    5     12.1       1.
 57   87    4     12.1      12.
 58   89    1     48.0     211.
 59   91    5     30.4       1.
 60   91    1     20.2      87.
 61   93    5     66.0       1.
 62   95    5     54.0       1.
 63  114    5     25.0       1.
 64   97    5     40.1       1.
 65   97    1     10.0      43.
 66  116    1     62.5     269.
 67  101    2     24.4     105.
 68  101    5     24.4       1.
 69  101    1     54.9     236.
 70  101    3     16.3      46.
 71  105    5     24.5       1.
 72  105    4     24.5      25.
 37  109    3      3.6       9.
TRANSPORT

               CINCINNATI-S  BLOODY  RUN  WATERSHED.  TRANSPORT
 117  140   38    1    5    5    2    3    4
      300.      .0001        10.
   1    1    1    1
   1    0    0    0   16
   2    1    0    0    1   394.0      4.5     2.60     .015
   3    2    0    0   16
   4    3    0    0    1   689.3      4.5     2.30     .015
   5    4    6    0   16
 7   8   0   0  16
 8   9   0   0   1    467.0     4.0    2.60    .015
 9   0   0   0  16
10   5   0   0   1    319.4     5.5    1.40    .015
11  10   0   0  16
12  11   0   0   1    345.3     6.0    0.90    .015
13  12   0   0  16
14  13   0   0   1    577.0     6.0    1.00    .015

15  14   0   0  16
16  15   0   0   1    462.7     6.0    1.40    .015
17  16   0   0  16
18  17   0   0   1   1428.5     6.0    1.80    .015
19  18   0   0  16
20  19   0   0   1   1641.0     6.5    1.25    .015
21  20   0   0  16
22  21   0   0   1    432.0     7.0    1.10    .015
23  22  25   0  16
24  25   0   0   1    149.7     5.0    1.00    .015
25   0   0   0  16
26  23   0   0   1   1238.5     7.5    1.30    .015
27  26  28   0  16
28  29   0   0   1    750.0     5.0    1.20    .015
29  30   0   0  16
30  31   0   0   1    700.0     5.0    1.00    .015
31   0   0   0  16
32  27   0   0   2    700.0     8.0    0.70    .015    10.0
33  32   0   0  16
34  33   0   0   2    900.0     8.0    0.70    .015    10.0
35  34   0   0  16
36  35   0   0   2    438.3     8.0    0.70    .015    10.0
37  36   0   0  16
38  37   0   0   2   1473.6     9.0    0.40    .015    11.0
39  38   0   0  16
40  39   0   0   2    350.0     9.0    0.40    .015    11.0
41  40  42   0  16
42  43   0   0   9   1673.1    8.63    0.30    .015
43  44 113   0  16
44  45   0   0   9   1433.4    8.63    0.28    .015
45  46  66   0  16
46  47   0   0   1    595.4     8.0    0.60    .015
47 110 111   0  16
48  49   0   0   1    291.2     4.5    1.30    .015
49  50  52   0  16
50  51   0   0   1    231.0     3.5    2.40    .015
51   0   0   0  16
52  53   0   0   1     76.0     3.0    3.40    .015
53   0   0   0  16
54  55   0   0   1    698.7     6.5    0.90    .015
55  56  58   0  16
56  57   0   0   1    209.5     3.5    2.10    .015
57   0   0   0  16
58  59   0   0   1    442.8     6.5    0.80    .015
59  60   0   0  16
60  61   0   0   1   1086.0     5.5    1.30    .015
61  62   0   0  16
62  63   0   0   1    463.4     5.0    1.50    .015
63  64   0   0  16
64  65   0   0   1    475.5     4.5    1.50    .015
65   0   0   0  16
66  67   0   0   1   1251.9     6.5    0.86    .015
67  68   0   0  16
68  69   0   0   1   [illegible]     6.0    1.00    .015
```

```
 69  70   0   0  16
 70  71   0   0   1   365.0     5.5     1.90    .015
 71  72   0   0  16
 72  73   0   0   1   404.7     5.5     2.30    .015
 73  74   0   0  16
 74  75   0   0   1   800.2     5.5     1.10    .015
 75  76  88   0  16
 76  77   0   0   1   481.0     4.0     1.57    .015


 77  78   0   0  16
 78  79   0   0   1   354.4     4.0     2.18    .015
 79  80   0   0  16
 80  81   0   0   1   253.6     3.5     5.70    .015
 81  82   0   0  16
 82  83   0   0   1   389.4     3.0     1.42    .015
 83  84   0   0  16
 84  85   0   0   1   342.0    2.75     1.67
 85  86   0   0  16
 86  87   0   0   1   325.9    2.25     1.65
 87   0   0   0  16
 88  89   0   0   1   858.0    4.00     2.10    .015
 89  90   0   0  16
 90  91   0   0   1  1700.0     3.5     1.50    .015
 91   0   0   0  16
 92  41   0   0  12  2190.0    9.25     0.28    .015    15.0            23.125
 93  92  94   0  16
 94  95   0   0   1   170.0     4.0     0.90    .015
 95   0   0   0  16
 96  93   0   0  12   775.0    9.25     0.32    .015    15.0            23.125
 97  96 115 117  16
 98  97   0   0  12   495.0    9.25     0.32    .015    15.0            23.125
 99  98 100   0  16
100 101   0   0   1   706.5     4.5     0.50    .015
101   0   0   0  16
102  99   0   0  12   500.0    9.25     0.32    .015    15.0            23.125
103 102   0   0  16
104 103   0   0  12  1556.0    9.25     0.34    .015    15.0            23.125
105 104   0   0  16
106 105   0   0  12   334.0    9.25     0.34    .015    15.0            23.125
107 106   0   0  16
108  54   0   0  16
109  40   0   0  16
110 109   0   0   1     1.0     8.0     2.00
111 108   0   0   1     1.0     8.0     2.00
112   0   0   0  16
113 112   0   0   1     1.0     8.0     2.00
114   0   0   0  16
115 114   0   0   1     1.0     8.0     2.00
116   0   0   0  16
117 116   0   0   1     1.0     8.0     2.00
 109  108   39   43  107
  51
 109  108   39   43  107
  2550.    0.0     0.0
 122   2550.
   0    0   42  222  567  880  942  812  645  314  108    0
      0.96       1.08       1.05       0.90       1.04       1.00       0.97
      1.00       1.00       1.00       1.00       1.00       1.00       1.00
      1.00       1.00       1.00       1.00       1.00       1.00       1.00
      0.74       0.67       0.63       0.59       0.54       0.56       0.67       0.96
      1.42       1.19       1.20       1.15       1.17       1.11       1.08       1.15
      1.21       1.23       1.25       1.21       1.17       1.15       0.88       1.07
      0.85       0.71       0.60       0.41       0.46       0.49       0.72       0.87
```

```
       0.77          1.57          1.02          0.87          0.91          0.94          1.07          1.07
       1.14          0.99          1.45          1.16          1.55          1.29          0.99          1.60
       1.05          1.45          1.10          0.50          0.66          1.33          1.10          0.88
       1.03          0.91          0.66          0.63          0.94          0.94          1.05          1.05
       1.16          0.94          1.33          1.22          1.44          1.10          0.88          1.05
       1.10          0.64          0.45          0.87          0.54          0.48          1.29          1.18
       1.37          1.49          1.30          1.12          0.89          0.58          0.45          0.67
       0.96          1.18          0.84          1.01          2.82          1.77          0.84          0.71


 -   71    2     0     7    07   30                50.000
    1  11                  105.6  9.3 365.  2.8 21.3                          8.3
    2  15                   52.8                                              8.3
    3  13                   17.6                                              8.3
    4  91                   34.3  9.7 115.  2.6 17.4                          7.6
    5  95                   34.3                                              7.6
    6 151                   58.6  8.7 172.  2.8 19.0                          7.6
    7 153                   14.6                                              7.6
    8 173                   14.8                                              7.6
    9 171                   44.2 13.5 133.  2.8 19.0                          7.6
   10 191                   38.0 15.2 111.  2.8 19.0                          7.6
   11 211                   20.0 14.0  59.  2.8 19.0                          7.6
   12 215                   13.4                                              7.6
   13 253                   69.0                                              7.6
   14 274                   43.0                                              7.6
   15 272                   10.7 14.7  60.  2.9 18.2                          7.6
   16 313                   37.5                                              7.4
   17 311                  212.7 18.21377.  2.6 17.3                          7.4
   19 331                   24.1 17.1 163.  2.6 17.3                          7.4
   20 332                   21.1 17.1 144.  2.6 17.3                          7.4
   21 353                   23.0                                              7.5
   22 352                   11.6 16.7  67.  2.8 17.4                          7.5
   23 351                    3.8 16.7  22.  2.8 17.4                          7.5
   24 371                   81.8 12.6 395.  2.7 17.1                          7.3
   25 395                   37.4                                              8.3
   26 393                   12.4                                              8.3
   27 412                   13.4 10.5  26.  4.5 22.7                          8.6
   28 413                    7.1                                              8.6
   291125                   43.2                                              8.6
   301123                   28.8                                              8.6
   31 431                   58.0 10.5  50.  4.5 22.7                          8.6
   32 433                   58.0                                              8.6
   33 453                   36.2                                              8.2
   34 455                   27.5                                              8.2
   35 452                   27.5 12.5 100.  3.9 20.8                          8.2
   361085                   14.3                                              8.6
   371093                    3.6                                              8.6
   38 511                   58.9 10.5 140.  4.5 22.7                          8.6
   39 531                   42.5  5.9 130.  4.5 22.7                          8.6
   40 571                   10.8 10.5  15.  4.1 31.3                         12.7
   41 575                   14.5                                             12.7
   42 573                   10.8                                             12.7
   43 592                   29.9  6.7  83.  4.5 22.7                          8.6
   44 595                   10.0                                              8.6
   45 611                   21.8  6.7  48.  3.3 26.3                         11.2
   46 613                   47.9                                             11.2
   47 612                   17.4  6.7  38.  3.3 26.3                         11.2
   48 652                   22.1 11.0  80.  3.1 19.2                          8.3
   49 651                   66.2 11.0 241.  3.1 19.2                          8.3
   50 715                   10.5                                              8.2
   51 714                   19.6                                              8.2
   52 754                   29.9                                              7.4
   53 772                   10.9 11.3  40.  3.0 17.3                          7.5
 55 871              36.1 11.6 139.  3.2 17.4                    7.6
 56 875              12.1                                        7.6
 57 874              12.1                                        7.6
 58 891              48.0 11.3 179.  3.0 17.3                    7.5
 59 915              30.4                                        7.6
 60 911              20.2-13.1  23.  3.7 18.5                    7.6
 61 935              66.0                                        8.6
 62 955              54.0                                        7.2


 631145              25.0                                        7.2
 64 975              40.1                                        7.2
 65 971              10.6  5.4  19.  2.9 17.0                    7.2
 661161              62.5  5.4 116.  2.9 17.0                    7.2
 671012              24.4  8.4  53.  4.0 21.0                    8.2
 681015              24.4                                        8.2
 691011              54.9  8.4 120.  4.0 21.0                    8.2
 701013              18.3                                        8.2
 711055              24.5                                        8.6
 721054              24.5                                        8.6
GRAPH
    9    1    0    5
  108  109   39   43  107
           GRAPHS  OF  CINCINNATI-S  BLOODY  RUN  WATERSHED
                           TIME  IN  HOURS
 Q - CFS
ENDPROGRAM
```

BLOODY RUN WATERSHED DATA
(WITH GUTTER-PIPE CARDS)

STORM OF NOVEMBER 14, 1970

```
CINCINNATI BLOODY RUN SEWER SYSTEM
     1     2375.80     13.50     10     2570.0     1     2570.14
NOVEMBER 14, 1970                       0.51
   0   8   8   9  10  10  11   0  12  12  14   0  15  15  16
   1   2   3   4  13
WATERSHED
 BLOODY RUN WATERSHED, CINCINNATI      COMBINED SEWER SYSTEM
      DATA FILE 3H5M04
   38  140 0730  5.0    3  42.
  120  5.0
.0001.0001.0001 0.08 0.08 0.070.0360.0480.036 0.12
 0.06 0.06 0.06 0.06 0.18 0.36 0.06 0.06 0.080.042
0.0360.0420.0420.0360.0420.0420.0360.0420.0420.036
 0.04 0.04 0.04 0.04 0.04 0.04 0.12 0.12 0.120.036
0.0360.048 0.07 0.08 0.08 0.07 0.08 0.080.0480.048
0.0480.0480.048 0.060.0240.0240.0240.0240.0240.024
0.0240.0240.0240.0240.0240.036 0.02 0.02 0.02 0.02
 0.02 0.02 0.02 0.02 0.02 0.02 0.02 0.02 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
 0.12 0.12 0.12 0.00 0.12 0.06 0.06 0.06 0.06 0.12
 0.12 0.12 0.12 0.06 0.06 0.24 0.06 0.06 0.06 0.06
 0.12 0.12 0.12 0.12 0.24 0.12 0.24 0.06 0.12 0.18
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 0.12 0.12 0.12 0.12 0.12 0.00 0.12 0.12 0.00 0.06
 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.12 0.06 0.06
 0.12 0.06 0.06 0.12 0.12 0.06 0.06 0.06 0.18 0.06
 0.06 0.06 0.06 0.06 0.00 0.06 0.06 0.06 0.06 0.06
 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06
 0.06 0.06 0.06 0.06 0.04 0.02 0.04 0.02 0.06 0.02
 0.04 0.06 0.06 0.06 0.06 0.06 0.05 0.02 0.01 0.01
 0.01 0.01 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
.0001.0001.0001.0001.0001.0001.0001.0001 0.12 0.12
 0.12 0.12 0.00 0.12 0.06 0.06 0.12 0.12 0.06 0.06
 0.12 0.00 0.00 0.00 0.00 0.00 0.12 0.12 0.12 0.12
```

```
0.00 0.06 0.06 0.06 0.06 0.12 0.12 0.12 0.06 0.06
0.12 0.12 0.12 0.12 0.00 0.12 0.06 0.06 0.06 0.06
0.12 0.06 0.00 0.00 0.06 0.12 0.00 0.00 0.00 0.00
0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.12 0.06
0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.00 0.00 0.00
0.00 0.00 0.00 0.00 0.00 0.00 0.12 0.12 0.12 0.12
0.12 0.06 0.00 0.00 0.00 0.00 0.00 0.00 0.06 0.06
0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.00 0.00
0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
   3    1  1591530.176.0 33.7.0310                      3.00 0.52    .00113
   3    2  1361138. 68.6 28.7.0384                      3.00 0.52    .00113
   3    3  1461262. 73.2 53.5.0442                      3.00 0.52    .00113
   3    4  1474112. 59.0 34.7.0301                      3.00 0.52    .00113
   3    5  1481558. 38.0 36.8.1120                      3.00 0.52    .00113
   3    6  145 970. 33.4 38.9.1050                      3.00 0.52    .00113
   3    7  143 481. 69.0 73.3.0315                      3.00 0.52    .00113
   2    8  1201701. 53.7 56.8.0902                      3.00 0.52    .00113
   3    9  1182389.250.2 51.9.0401                      3.00 0.52    .00113
   2   10  1212625. 45.2 58.5.0200                      3.00 0.52    .00113
   2   11  1221487. 38.4 81.4.0430                      3.00 0.52    .00113
   2   12  1232591. 81.8 54.5.0320                      3.00 0.52    .00113
   2   13   391523. 49.8 10.8.0500                      3.00 0.52    .00113
   2   14  124 380. 20.2 34.7.0350                      3.00 0.52    .00113
   2   15  1271167. 72.0 20.7.0370                      3.00 0.52    .00113
   2   16  1285515.116.0 45.6.0375                      3.00 0.52    .00113
   2   17  1291598. 91.2 64.5.0238                      3.00 0.52    .00113
   2   18  108 830. 14.3 24.5.0400                      3.00 0.52    .00113
   2   19  1302053. 58.9 39.8.0220                      3.00 0.52    .00113
   2   20  1311646. 42.5 39.7.0150                      3.00 0.52    .00113
   2   21  1331798. 36.1 42.3.0420                      3.00 0.52    .00113
   2   22  1341545. 39.9 46.4.0250                      3.00 0.52    .00113
   2   23  1493571. 87.1 60.8.0570                      3.00 0.52    .00113
   2   24  1512461. 88.3 38.7.0630                      3.00 0.52    .00113
   2   25  140 839. 30.1 44.6.0830                      3.00 0.52    .00113
   2   26  1391310. 29.9 71.7.0640                      3.00 0.52    .00113
   2   27  1411072. 21.9 56.4.0590                      3.00 0.52    .00113
   3   28  1422335. 60.3 48.7.0430                      3.00 0.52    .00113
   2   29  1382091. 48.0 39.8.0360                      3.00 0.52    .00113
   3   30  1371954. 50.6 29.7.0660                      3.00 0.52    .00113
   2   31  1251243. 66.0  5.0.0495                      3.00 0.52    .00113
   2   32  126 904. 54.0  4.9.0250                      3.00 0.52    .00113
   1   33  154 725. 25.0 16.8.0270                      3.00 0.52    .00113
   1   34  153 896. 50.1 10.9.0235                      3.00 0.52    .00113
   1   35  155 838. 62.5 39.5.0125                      3.00 0.52    .00113
   1   36  1567600.122.0 49.7.0659                      3.00 0.52    .00113
   1   37  1581138. 49.0 33.8.0710                      3.00 0.52    .00113
   2   38  109 210.  3.6 95.0.0400                      3.00 0.52    .00113

   3 118    31   2    5.50  123.00    .0133     0.0     0.0    .012
   2 120    27   2    3.00  950.00    .0350     0.0     0.0    .012
   2 121    33   2    2.00  321.00    .0330     0.0     0.0    .012
   2 122    35   2    2.50  250.00    .0350     0.0     0.0    .012
   2 123    37   2    2.25  295.00    .0078     0.0     0.0    .012
   2 124    41   2    2.00  381.00    .0024     0.0     0.0    .012
   2 125    93   2    2.00 1166.00    .0100     0.0     0.0    .012
   2 126    95   2    2.50  200.00    .0700     0.0     0.0    .012
   2 127   112   2    3.50  220.00    .0170     0.0     0.0    .012
   2 128    43   2    2.00   84.00    .0093     0.0     0.0    .012
   2 129    45   2    2.00  450.00    .0032     0.0     0.0    .012
   2 130    51   2    4.00  820.00    .0093     0.0     0.0    .012
   2 131    53   2    2.50  720.00    .0110     0.0     0.0    .012
   2 133    57   2    3.00  190.00    .0342     0.0     0.0    .012
```

```
    2  134   59    2     3.00  340.00    .0103      0.0       0.0     .012
    3  136    9    2     3.25  321.00    .0150      0.0       0.0     .012
    3  137   91    2     3.50  372.00    .0001      0.0       0.0     .012
    2  138   89    2     2.00   51.20    .0070      0.0       0.0     .012
    2  139   75    2     0.83  250.00    .0400      0.0       0.0     .012
    2  140   71    2     1.25  280.00    .0130      0.0       0.0     .012
    2  141   77    2     1.75  241.00    .0105      0.0       0.0     .012
    3  142   87    2     1.00  190.00    .0250      0.0       0.0     .012
    3  143   25    2     4.00  227.00    .0150      0.0       0.0     .012
    3  145   21    2     1.75  239.00    .0056      0.0       0.0     .012
    3  146   15    2     1.50  158.00    .0386      0.0       0.0     .012
    3  147   17    2     2.50  250.00    .0500      0.0       0.0     .012
    3  148   19    2     1.50  100.00    .0600      0.0       0.0     .012
    2  149   61    2     1.50  255.00    .0283      0.0       0.0     .012
    2  151   65    2     3.50  400.00    .0450      0.0       0.0     .012
    1  153   97    2     2.00  500.00    .0225      0.0       0.0     .012
    1  154  114    2     2.00  165.60    .0081      0.0       0.0     .012
    1  155  116    2     2.75  400.00    .0460      0.0       0.0     .012
    1  156  101    2     1.00  650.00    .0500      0.0       0.0     .012
    1  158  105    2     3.50  300.00    .0091      0.0       0.0     .012
    3  159    1    2     3.50 1400.00    .0300      0.0       0.0     .012

   38
    1    9   15   17   19   21   25   27   31   33   35   37   39   41   43  112
   45   51   53  108   57   59   61   65   71   75   77   87   89   91   93   95
  114   97  116  101  105  109
    0    0
    0    0    0
   71   38  5.0   07   30  140    0
       10.        21.   1
        1.        125.      100.
    1    1    1      105.6      454.
    2    1    5       52.8        1.
    3    1    3       17.6       47.
    4    9    1       34.3      147.
    5    9    5       34.3        1.
    6   15    1       58.6      252.
    7   15    3       14.6       38.
    8   17    3       14.8       39.
    9   17    1       44.2      190.
   10   19    1       38.0      164.
   11   21    1       20.0       86.
   12   21    5       13.4        1.
   13   25    3       69.0      180.
   14   27    4       43.0       43.
   15   27    2       10.7       46.
   16   31    3       37.5       98.
   17   31    1      212.7      916.
   19   33    1       24.1      117.
   20   33    2       21.1       78.
   21   35    3       23.0       60.
   22   35    2       11.6       50.
   23   35    1        3.8       16.
   24   37    1       81.8      352.
   25   39    5       37.4        1.
   26   39    3       12.4       32.
   27   41    2       13.4       58.
   28   41    3        7.1       19.
   31   43    1       58.0      250.
   32   43    3       58.0      151.
   29  112    5       43.2        1.
   30  112    3       28.8       75.
```

```
     33   45    3       36.2        96.
     34   45    5       27.5         1.
     35   45    2       27.5       108.
     38   51    1       58.9       254.
     39   53    1       42.5       183.
     36  108    5       14.3         1.
     40   57    1       10.8        46.
     41   57    5       14.5         1.
     42   57    3       10.8        28.
     43   59    2       29.9       129.
     44   59    5       10.0         1.
     45   61    1       21.8        94.
     46   61    3       47.9       125.
     47   61    2       17.4        75.
     48   65    2       22.1        95.
     49   65    1       66.2       285.
     50   71    5       10.5         1.
     51   71    4       19.6        20.
     52   75    4       29.9        30.
     53   77    2       10.9        47.
     54   77    1       11.0        47.
     55   87    1       36.1       155.
     56   87    5       12.1         1.
     57   87    4       12.1        12.
     58   89    1       48.0       211.
     59   91    5       30.4         1.
     60   91    1       20.2        87.
     61   93    5       66.0         1.
     62   95    5       54.0         1.
     63  114    5       25.0         1.
     64   97    5       40.1         1.
     65   97    1       10.0        43.
     66  116    1       62.5       269.
     67  101    2       24.4       105.
     68  101    5       24.4         1.
     69  101    1       54.9       236.
     70  101    3       18.3        48.
     71  105    5       24.5         1.
     72  105    4       24.5        25.
     37  109    3        3.6         9.
 TRANSPORT
 1                 CINCINNATI-S  BLOODY  RUN  WATERSHED.  TRANSPORT
   117  140    38    1     5     5    2    3    4
        300.      .0001         10.
     1     1     1     1
     1   0   0   0  16
     2   1   0   0   1   394.0      4.5     2.60     .015
     3   2   0   0  16
     4   3   0   0   1   689.3      4.5     2.30     .015
     5   4   6   0  16
     6   7   0   0   1   380.4      4.0     2.20     .015
     7   8   0   0  16
     8   9   0   0   1   467.0      4.0     2.60     .015
     9   0   0   0  16
    10   5   0   0   1   319.4      5.5     1.40     .015
    11  10   0   0  16
    12  11   0   0   1   345.3      6.0     0.90     .015
    13  12   0   0  16
    14  13   0   0   1   577.0      6.0     1.00     .015
    15  14   0   0  16
    16  15   0   0   1   462.7      6.0     1.40     .015
```

```
17  16   0   0  16
18  17   0   0   1  1428.5    6.0   1.80   .015
19  18   0   0  16
20  19   0   0   1  1641.0    6.5   1.25   .015
21  20   0   0  16
22  21   0   0   1   432.0    7.0   1.10   .015
23  22  25   0  16
24  25   0   0   1   149.7    5.0   1.00   .015
25   0   0   0  16
26  23   0   0   1  1238.5    7.5   1.30   .015
27  26  28   0  16
28  29   0   0   1   750.0    5.0   1.20   .015
29  30   0   0  16
30  31   0   0   1   700.0    5.0   1.00   .015
31   0   0   0  16
32  27   0   0   2   700.0    8.0   0.70   .015   10.0
33  32   0   0  16
34  33   0   0   2   900.0    8.0   0.70   .015   10.0
35  34   0   0  16
36  35   0   0   2   438.3    8.0   0.70   .015   10.0
37  36   0   0  16
38  37   0   0   2  1473.6    9.0   0.40   .015   11.0
39  38   0   0  16
40  39   0   0   2   350.0    9.0   0.40   .015   11.0
41  40  42   0  16
42  43   0   0   9  1673.1   8.63   0.30   .015
43  44 113   0  16
44  45   0   0   9  1433.4   8.63   0.28   .015
45  46  66   0  16
46  47   0   0   1   595.4    8.0   0.60   .015
47 110 111   0  16
48  49   0   0   1   291.2    4.5   1.30   .015
49  50  52   0  16
50  51   0   0   1   231.0    3.5   2.40   .015
51   0   0   0  16
52  53   0   0   1    76.0    3.0   3.40   .015
53   0   0   0  16
54  55   0   0   1   698.7    6.5   0.90   .015
55  56  58   0  16
56  57   0   0   1   209.3    3.5   2.10   .015
57   0   0   0  16
58  59   0   0   1   442.8    6.5   0.80   .015
59  60   0   0  16
60  61   0   0   1  1086.0    5.5   1.30   .015
61  62   0   0  16
62  63   0   0   1   463.4    5.0   1.50   .015
63  64   0   0  16
64  65   0   0   1   475.5    4.5   1.50   .015
65   0   0   0  16
66  67   0   0   1  1251.9    6.5   0.86   .015
67  68   0   0  16
68  69   0   0   1   930.4    6.0   1.00   .015
69  70   0   0  16
70  71   0   0   1   365.0    5.5   1.90   .015
71  72   0   0  16
72  73   0   0   1   404.7    5.5   2.30   .015
73  74   0   0  16
74  75   0   0   1   800.2    5.5   1.10   .015
75  76  88   0  16
76  77   0   0   1   481.0    4.0   1.57   .015
77  78   0   0  16
78  79   0   0   1   354.4    4.0   2.18   .015
```

```
 79  80   0   0  16
 80  81   0   0   1   253.6     3.5    5.70    .015
 81  82   0   0  16
 82  83   0   0   1   389.4     3.0    1.42    .015
 83  84   0   0  16
 84  85   0   0   1   342.0    2.75    1.67
 85  86   0   0  16
 86  87   0   C   1   325.9    2.25    1.65
 87   0   0   0  16
 88  89   0   0   1   858.0    4.00    2.10    .015
 89  90   0   0  16
 90  91   0   0   1  1700.0     3.5    1.50    .015
 91   0   0   0  16
 92  41   0   0  12  2190.0    9.25    0.28    .015    15.0            23.125
 93  92  94   0  16
 94  95   0   0   1   17C.0     4.0    0.90    .015
 95   0   0   0  16
 96  93   0   0  12   775.0    9.25    0.32    .015    15.0            23.125
 97  96 115 117  16
 98  97   0   0  12   495.0    9.25    0.32    .015    15.0            23.125
 99  98 100   C  16
100 101   0   U   1   706.5     4.5    0.50    .015
101   0   0   0  16
102  99   0   0  12   500.C    9.25    0.32    .015    15.0            23.125
103 102   0   0  16
104 103   0   G  12  1556.0    9.25    0.34    .015    15.0            23.125
105 104   0   0  16
106 105   C   C  12   334.0    9.25    0.34    .015    15.0            23.125
107 106   0   0  16
108  54   0   0  16
109  48   0   0  16
110 109   0   0   1     1.0     8.0    2.00
111 108   0   0   1     1.0     8.0    2.00
112   0   0   U  16
113 112   0   0   1     1.0     8.0    2.00
114   0   0   0  16
115 114   0   0   1     1.0     8.0    2.00
116   0   0   0  16
117 116   0   0   1     1.0     8.0    2.00
  109  108    39    43   107
   51
  109  108    39    43   107
   2550.      0.0        0.0
  122    2550.
    0     0    42   222   567   880   942   812   645   314   108     0
         0.96        1.08        1.05        0.90        1.04        1.00        0.97
         1.00        1.00        1.00        1.00        1.00        1.00        1.00
         1.00        1.00        1.00        1.00        1.00        1.00        1.00
         0.74        0.67        0.63        0.59        0.54        0.56        0.67        0.96
         1.42        1.19        1.20        1.15        1.17        1.11        1.08        1.15
         1.21        1.23        1.25        1.21        1.17        1.15        0.88        1.07
         0.85        0.71        0.60        0.41        0.46        0.49        0.72        0.87
         0.77        1.57        1.02        0.87        0.91        0.94        1.07        1.07
         1.14        0.99        1.45        1.16        1.55        1.29        0.99        1.60
         1.05        1.05        1.10        0.50        0.66        1.33        1.10        0.88
         1.03        0.91        0.66        0.63        0.94        0.94        1.05        1.05
         1.16        0.94        1.33        1.22        1.44        1.10        0.88        1.05
         1.10        0.64        0.45        0.87        0.54        0.48        1.29        1.18
         1.37        1.49        1.30        1.12        0.89        0.58        0.45        0.67
         0.96        1.18        0.84        1.01        2.82        1.77        0.84        0.71
   71     2     0     7    07    30                  50.000
  1  11                   105.6   9.3 365.   2.8 21.3                        8.3
```

```
                                                                   8.3
  2  15                    52.8                                    8.3
  3  13                    17.6                                    7.6
  4  91                    34.3  9.7 115.  2.6 17.4                7.6
  5  95                    34.3                                    7.6
  6 151                    58.6  8.7 172.  2.8 19.0                7.6
  7 153                    14.6                                    7.6
  8 173                    14.8                                    7.6
  9 171                    44.2 13.5 133.  2.8 19.0                7.6
 10 191                    38.0 15.2 111.  2.9 19.0                7.6
 11 211                    20.0 14.0  59.  2.8 19.0                7.6
 12 215                    13.4                                    7.6
 13 253                    69.0                                    7.6
 14 274                    43.0                                    7.6
 15 272                    10.7 14.7  60.  2.9 18.2                7.4
 16 313                    37.5                                    7.4
 17 311                   212.7 18.21377.  2.6 17.3                7.4
 19 331                    24.1 17.1 163.  2.6 17.3                7.4
 20 332                    21.1 17.1 144.  2.6 17.3                7.5
 21 353                    23.0                                    7.5
 22 352                    11.6 16.7  67.  2.8 17.4                7.5
 23 351                     3.8 16.7  22.  2.8 17.4                7.3
 24 371                    81.8 12.6 395.  2.7 17.1                8.3
 25 395                    37.4                                    8.3
 26 393                    12.4                                    8.6
 27 412                    13.4 10.5  26.  4.5 22.7                8.6
 28 413                     7.1                                    8.6
 291125                    43.2                                    8.6
 301123                    28.8                                    8.6
 31 431                    58.0 10.5  50.  4.5 22.7                8.6
 32 433                    58.0                                    8.2
 33 453                    36.2                                    8.2
 34 455                    27.5                                    8.2
 35 452                    27.5 12.5 100.  3.9 20.8                8.6
 361085                    14.3                                    8.6
 371093                     3.6                                    8.6
 38 511                    58.9 10.5 140.  4.5 22.7                8.6
 39 531                    42.5  5.9 130.  4.5 22.7               12.7
 40 571                    10.8 10.5  15.  4.1 31.3               12.7
 41 575                    14.5                                   12.7
 42 573                    10.8                                    8.6
 43 592                    29.9  6.7  83.  4.5 22.7                8.6
 44 595                    10.0                                   11.2
 45 611                    21.8  6.7  48.  3.3 26.3               11.2
 46 613                    47.9                                   11.2
 47 612                    17.4  6.7  38.  3.3 26.3                8.3
 48 652                    22.1 11.0  80.  3.1 19.2                8.3
 49 651                    66.2 11.0 241.  3.1 19.2                8.2
 50 715                    10.5                                    8.2
 51 714                    19.6                                    7.4
 52 754                    29.9                                    7.5
 53 772                    10.9 11.3  40.  3.0 17.3                7.5
 54 771                    11.0 11.3  41.  3.0 17.3                7.6
 55 871                    36.1 11.6 139.  3.2 17.4                7.6
 56 875                    12.1                                    7.6
 57 874                    12.1                                    7.5
 58 891                    48.0 11.3 179.  3.0 17.3                7.6
 59 915                    30.4                                    7.6
 60 911                    20.2 13.1  23.  3.7 18.5                8.6
 61 935                    66.0                                    7.2
 62 955                    54.0                                    7.2
 631145                    25.0                                    7.2
 64 975                    40.1
```

```
  65 971                10.0  5.4  19.   2.9 17.0                      7.2
  661161                62.5  5.4 116.   2.9 17.0                      7.2
  671012                24.4  8.4  53.   4.0 21.0                      8.2
  681015                24.4                                           8.2
  691011                54.9  8.4 120.   4.0 21.0                      8.2
  701013                18.3                                           8.2
  711055                24.5                                           8.6
  721054                24.5                                           8.6
 GRAPH
    9     1     0     5
  108   109    39    43   107
              GRAPHS   OF   CINCINNATI-S  BLOODY   RUN   WATERSHED
                                  TIME   IN   HOURS

  Q - CFS
- ENDPROGRAM
```

BANK WATERSHED DATA
(WITH GUTTER-PIPE CARDS)

STORM OF AUG. 4, 1971

```
CINCINNATI BLOODY RUN SEWER SYSTEM)BANK*
   1      105.0       .53   10     241.8    1    241.8
AUG. 4, 1971                 .45
   0   8   8   9  10  10  11   0  12  12  14   0  15  15  16
   1   2   3   4  13
WATERSHED
BLOODY RUN WATERSHED (BANK) , CINCINNATI COMBINED SEWER SYSTEM
  DATA FILE B1H5M28
   3  85 0915  5.0     1  40.
  67 5.0
  .21  .21  .21  .15  .15  .15  .21  .21  .21  .24
  .24  .24  .12  .24  .12  .18  .18  .18  .09  .09
  .11  .24  .24  .24  .24  .12  .12  .00  .00  .00
  .00  .00  .00  .00  .00  .00  .00  .00  .00  .00
  .00  .00  .00  .00  .00  .00  .00  .00  .00  .00
  .00  .00  .00  .00  .00  .00  .00  .06  .06  .06
  .06  .06  .06  .06  .06  .06  .06
    1  19     5  32.  .92 39.8.0220                                   .00113
    1   1    96  32.  .92 39.8.0220                                   .00113
    1   2    97  32.  .92 39.8.0220                                   .00113
    1   3    98  32.  .92 39.8.0220                                   .00113
    1   4    99  32.  .92 39.8.0220                                   .00113
    1   5   100  32.  .92 39.8.0220                                   .00113
    1   6   101  32.  .92 39.8.0220                                   .00113
    1   7   102  32.  .92 39.8.0220                                   .00113
    1   8   103  32.  .92 39.8.0220                                   .00113
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    1  22    85  32.  .92 39.8.0220                                   .00113
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   1   81   80    2    2.5       145.   .0120                           .012
   1   82   77    2    1.25      162.   .0222                           .012
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   1   88   87    2    1.5        60.   .0092                           .012
   1   89   87    2    1.        165.   .0100                           .012
   1   90   89    2    1.        166.   .0109                           .012
   1   91   81    2    1.        161.   .0181                           .012
   1   92   91    2    1.        160.   .0059                           .012
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   1   94   93    2     .67      201.   .0073                           .012
   1   95   94    2     .67      219.   .0079                           .012
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   1  100   99    2    1.25      369.   .0054                           .012
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   1  104  103    2     .67      217.   .0023                           .012
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   1  112  111    2    1.         60.   .0100                           .012

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        1.       125.        100.
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    94   27   1       1.41       6.1
    95   27   1       1.41       6.1
TRANSPORT

1  CINCINNATI BLCODY RUN WATERSHED  (BANK) . TRANSPORT
    9   85    3    1    3    3    2    3    4
    300.       .0001         0
    1    1    1    1
   1   2   0   0  16
   2   7   0   0   1   161.0     4.5    1.38    .015
   9   3   0   0   1   161.0     4.5    1.38    .015
   3   4  26   0  16
   4   5   0   0   1   242.0     4.0    2.09    .015
   5   0   0   0  16
   7   9   0   0  16
  26  27   0   0   1    61.0     3.0    2.70    .015
  27   0   0   0  16
    1    5   27
    3
    1    5   27
   2550.     0.0       0.0
   20   2550.
    0    0   42  222  567  880  942  812  645  314  108    0
      0.96      1.08      1.05      0.90      1.04      1.00      0.97
      1.00      1.00      1.00      1.00      1.00      1.00      1.00
      1.00      1.00      1.00      1.00      1.00      1.00      1.00
      0.74      0.67      0.63      0.59      0.54      0.56      0.67      0.96
      1.42      1.19      1.20      1.15      1.17      1.11      1.08      1.15
      1.21      1.23      1.25      1.21      1.17      1.15      0.88      1.07
      0.85      0.71      0.60      0.41      0.46      0.49      0.72      0.87
      0.77      1.57      1.02      0.87      0.91      0.94      1.07      1.07
      1.14      0.99      1.45      1.16      1.55      1.29      0.99      1.60
      1.05      1.05      1.10      0.50      0.66      1.33      1.10      0.88
      1.03      0.91      0.66      0.63      0.94      0.94      1.05      1.05
      1.16      0.94      1.33      1.22      1.44      1.10      0.88      1.05
      1.10      0.64      0.45      0.87      0.54      0.48      1.29      1.18
      1.37      1.49      1.30      1.12      0.89      0.58      0.45      0.67
      0.96      1.18      0.84      1.01      2.82      1.77      0.84      0.71
   95    2    0    6    9   15                1.37
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   1  51                   .9 10.5   3. 4.53 22.7  20.               8.61  1
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  5  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
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 57  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 58  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 59  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 60  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 61  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 62  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 63  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 64  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 65  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 66  51                  .9 10.5    3. 4.53 22.7  20.                 8.61  1
 20 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
```

```
   68 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   69 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   70 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   71 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   72 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   73 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   74 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   75 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   76 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   77 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   78 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   79 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   80 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   81 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   82 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   83 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   84 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   85 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   86 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   87 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   88 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   89 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   90 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   91 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   92 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   93 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   94 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   95 271                 1.4  5.9    4. 4.53 22.7  20.                 8.61  1
   38  73
  GRAPH
      9    1    0    3
      5   27    1
         PLOT OF BANK DISCHARGE - BLOODY RUN WATERSHED
                           TIME (HOURS)
  RUNOFF     (CFS)
  ENDPROGRAM
```

APPENDIX E

TYPICAL INPUT AND OUTPUT

The purpose of this appendix is to present to the reader some typical input and output from the EPA SWMM. The comparisons in Appendix C were all derived from data in this form.

The storm used here occurred on April 19, 1972. Since no quality data were collected for this storm, the computed pollutographs are not given (though this and much other information was actually calculated by the computer).

With the information given here, the reader can make his own comparisons between field data and the computed hydrographs.

INPUT: RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: April 19, 1972

Time	Intensity of Rain (in/hr)			Discharge cfs
	Laidlaw Gage	Woodward Gage	Ridge Gage	
MONITORING STATION: Bank 54"				
1700:00	0.12	0.24	0.24	9.33
1707:30	0.12	0.24	0.36	6.90
1952:30	-	-	-	11.25
2000:00	-	-	-	16.2
2007:30	-	-	-	11.25
2015:00	0.12	0.0	0.12	6.90
MONITORING STATION: Bank 78"				
2000:00	0.24	0.24	0.12	18.6
2007:30	0.18	0.12	0.12	18.6
2015:00	-	-	0.12	15.26
2022:30	-	-	0.12	15.26
2030:00	-	-	-	9.9
2037:30	-	-	-	7.09
2045:00	-	-	-	6.08
MONITORING STATION: Outlet				
1615:00	0.30	1.12	0.12	75.1
1622:30	0.18	0.12	0.18	104.4
1630:00	0.12	0.12	0.06	104.4
1637:30	0.12	0.06	0.030	127.7
1645:00	0.12	0.0	0.24	297.9
1652:30	0.12	0.30	0.24	280.0
1700.00	0.12	0.24	0.24	259.0
1707:30	0.12	0.24	0.36	251.5
1715:00		0.0	0.60	251.5
1722:30				380.9
1730:00				363.5
1737:30				330.2
1745:00				266.7
1752:30				221.7
1800:00				178.1
1807:30				127.7
1815:00				75.1
1822:30				58.2
2015:00	0.12	0.0	0.12	313.8
2022:30			0.12	397.9
2030:00				347.0
2037:30				313.8
2045:00				266.7
2052:30				221.7
2100:00				199.0
2107:30				127.7
2115:00				104.4
2122:30				67.73

OUTPUT: COMPUTED HYDROGRAPHS AT FIVE LOCATIONS

```
                                       SELECTED OUTFLOW HYDROGRAPHS - CFS
EXTERNAL
ELEMENT    TIME STEP
NUMBER        1            2           3           4           5           6           7           8           9          10

 109     [illegible]     2.377       7.323       5.444       6.775       4.582       3.555       2.631       2.067       1.645
            1.355        1.147       1.139       1.483       1.977       2.169       2.428  [illegible]      2.784       2.908
            2.663   [illegible]      3.114  [illegible]      2.968       2.519       2.102       1.852       1.645       1.502
            1.383        1.236       1.226       1.165       1.135       1.079       1.064       1.038       2.017       4.210
            4.456        3.836       5.419       7.263       5.592       4.757       5.074       7.918      12.222      11.961
           10.316        4.627       5.735  [illegible]      2.494       2.305       1.816       1.483       1.240       1.063
       [illegible]  [illegible]      0.712       0.638       0.577       0.527       0.485       0.453       0.420       0.395
       [illegible]  [illegible]      0.339       0.326       0.313       0.302       0.292       0.283       0.275       0.268
       [illegible]  [illegible]      4.728       5.665      11.480      10.652       9.862       8.769       6.835       5.611
            4.559        3.353       2.545       1.591       1.603       1.326       1.137       0.970       0.843       0.741
       [illegible]  [illegible]      3.543       0.455       0.457       0.425       0.358       0.375       0.354       0.335
       [illegible]  [illegible]      0.295       0.265       0.276       0.269       0.260  [illegible]      0.248       0.242

 108   [illegible]  [illegible]     13.958      27.348  [illegible]     16.935      10.789       6.845       6.321       5.159
       [illegible]       3.311  [illegible]      3.132  [illegible]      5.343       6.541       7.473       8.006       8.357
       [illegible]       8.342       9.127       9.115       8.932       8.052       6.733       5.736       4.583       4.422
       [illegible]  [illegible]      3.512       3.335       3.158       3.056       3.014       2.953       3.941       8.211
       [illegible]      13.371      13.775      18.326      20.562      16.113      13.678      18.207      30.150      36.615
       [illegible]  [illegible]  [illegible]     13.177  [illegible]      6.712       5.367       4.257       3.445       2.856
       [illegible]       2.363       2.054       1.828       1.622       1.472       1.380       1.281       1.168       1.100
       [illegible]  [illegible]  [illegible]      0.914       0.864       0.857       0.833       0.812       0.793       0.776
       [illegible]       1.217  [illegible]     21.744  [illegible]     34.459      30.014      26.633      22.057      17.193
       [illegible]  [illegible]  [illegible]  [illegible]      4.817       3.815       3.151       2.664       2.356       2.169
       [illegible]  [illegible]      1.834       1.359       1.289       1.158       1.123       1.060       1.006       0.960
       [illegible]  [illegible]  [illegible]  [illegible]  [illegible]      0.765       0.768       0.752       0.738       0.725

  36        3.111        4.272       9.769      21.759      31.684      31.712      27.757      24.155      21.333      18.287
           15.866       13.957      12.153      11.528      11.783      12.722      14.200      15.885      17.596      19.155
           20.317   [illegible]     23.423  [illegible]     19.182      17.631      16.065      14.400      13.070      12.076
       [illegible]  [illegible]  [illegible]     14.244      26.255      34.796      36.503      32.556      29.351      29.756
       [illegible]  [illegible]  [illegible]     57.122  [illegible]    161.657      65.878      88.074      55.284      55.335
       [illegible]      52.441  [illegible]     74.514      61.423      50.592      42.251      35.800      30.635      27.056
       [illegible]  [illegible]     19.451      17.657      16.115      14.683      13.457      12.465      11.602      10.850
       [illegible]       9.565       9.136       8.759       8.378       8.221       7.651       7.369       7.054       6.821
       [illegible]  [illegible]  [illegible]  [illegible]     37.686      57.436      72.536      76.176      70.175      62.920
       [illegible]      51.577      44.495      41.151      34.326      29.488      25.813      22.612      20.245      18.371
           16.725       15.173      13.831      12.721      11.775      10.566      10.276       9.685       9.170       8.718
            8.322        7.977       7.651       7.337       7.035       6.755       6.497       6.263       6.050       5.856

  43        3.652        8.532      23.746      37.253      53.645      62.417      53.647      39.202      30.165      25.361
           16.444       13.715      11.993      11.721      12.605      14.719      17.311      20.200      22.917      24.891
           25.278   [illegible]     27.463      27.548      26.505      25.249      22.888      20.078      17.649      15.603
       [illegible]      12.955  [illegible]     11.649      12.569      15.358      17.258      16.005      16.545      22.411
       [illegible]      33.218      43.232  [illegible]     64.364      67.224      57.465      58.382      76.695      58.514
       [illegible]  [illegible]  [illegible]  [illegible]  [illegible]  [illegible]  [illegible]  [illegible]  [illegible]  [illegible]
```

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OUTPUT: COMPUTED HYDROGRAPHS AT FIVE LOCATIONS
(continued)

```
       [illegible]
       11.432    9.933    8.785    7.005    7.341    6.800    6.265    5.766    5.336    4.938
        4.671    4.449    4.185    3.554    3.828    3.684    3.556    3.444    3.345    3.256
        3.174    5.491   14.242   32.745   57.459   90.958  100.352   88.053   74.940   62.341
       54.115   47.757   35.717   27.755   22.884   17.661   14.753   12.318   10.521    9.215
        8.373    7.633    6.955    6.448    5.939    5.477    5.088    4.759    4.478    4.237
        4.031    3.852    3.657    3.549    3.435    3.333    3.239    3.156    3.081    3.014

107     8.565    9.546   16.324   28.267   37.666   43.201   55.111   67.665   85.580  115.640
      121.048  120.174 [illegible] 57.591  74.431   43.165   30.854   38.584   32.647   34.644
       37.696   61.323   43.857   46.645   47.655   48.084   46.048   48.049   47.953   47.953
       43.140   35.637   37.456   37.456   40.351   40.991   40.551   41.221   51.567   57.467
       58.100   40.355   43.575   48.744   75.643   92.212  161.552  189.546  170.860  180.556
      184.353  241.867  237.861  194.435  171.232  147.151  126.036  110.659  101.850   94.781
       52.645   55.432   57.674   36.623   39.425   36.211   33.045   31.016   28.654   27.145
       25.439   24.153   22.978   22.026   21.366   20.221   19.634   19.055   18.635   18.038
       17.478   17.479   17.678   19.459   26.417   42.314   65.335   80.787  152.538  156.117
       15.044  135.047  127.281  105.761  103.921   53.506   87.717   87.356   89.002   76.578
       33.376   30.457   34.484   31.484   30.667   27.493   26.526   25.044   23.771   22.573
       21.549   20.582   19.565   19.325   18.913   18.201   17.596   17.188   16.771   16.372
```

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NOTES

The above storm hydrographs were computed at 5-minute intervals beginning at 1:00 PM, April 19, 1972.

The numbers 109, 108, 39, 43 and 107 indicate the Bank 54", Bank 78", Longview 11' x 9', Longview 12' x 9' and Outlet monitoring stations respectively.